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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XXVIII. No. 10. OCTOBER, 1933.

EDITORIAL NOTES.

Cotton Fabric in Concrete Road Construction.

A METHOD of road construction has been developed in the United States in which the two courses of concrete are separated by cotton or linen fabric. The bottom course is a relatively weak mixture; upon this, before the concrete has set, the fabric is laid and tamped sufficiently to permit of mortar passing through the openings in the fabric and incorporating the latter in the concrete. The top course is a rich mixture containing small aggregate, and is laid immediately after the placing of the fabric.

Experiments have been carried out by the Ministry of Transport's Technical Advisory Committee to determine whether difficulties are encountered during construction, and to investigate the claim made for the method that good adhesion can be obtained between the two courses, and that the top course can be removed and replaced without damage to the bottom course. Six test slabs, each 10 ft. square, were laid in the grounds of the Experimental Station, Harmondsworth, where they were not subjected to traffic. The results are published in "Experimental Work on Roads" (London: H.M. Stationery Office. Price 1s. 6d. net). The best bond was obtained with open mesh material, but wherever fabric was used it was easy to separate the courses, and the bond was weaker than in the two-course slab without fabric. The tests do not indicate how the top courses would behave if they were to carry traffic, but the general result of the experiment is to show that the use of cotton and linen fabrics in the manner described is practicable and likely to attain the objects claimed. It does not seem, however, that any practical gain, economic or otherwise, is likely to result from the use of this method in the normal practice of concrete road construction, since experience in this country indicates that concrete roads rarely fail as a result of abrasion.

From the conclusion just quoted from the Report of the Ministry's Committee it would appear that the real object of adopting this type of concrete road has been overlooked. Fabric is not used to enable the wearing surface of slabs which are badly abraded to be replaced—abrasion, as the Report states, rarely causes failure. When fabric is inserted between the slabs the object is to facilitate the replacement of cracked portions. Experiments have been made for several years at a London works' entrance where the roadway was laid as two-course concrete with an intervening layer of fabric and is continuously subjected to heavy lorry traffic. At these works sections of the wearing surface about 1 sq. yd. in area have been cut out after 12 or 18 months' wear, and have

been replaced. It has been found that the surfacing is easily removed without pneumatic tools, this operation being little more difficult than the removal of an asphalt coating. The concrete can also be replaced so that it is scarcely distinguishable from the original surface.

We understand that this method of construction is now being adopted on a main road in the London area, and that it has been applied on the embankment leading to a new bridge in South Wales. In the latter case it is intended to remove the top course when the filling has setttled and replace the concrete by a greater thickness which will raise the level of the road surface to that of the carriageway on the bridge, thus reducing the cost of the adjustment that is always required in such cases.

Piling at Cherbourg Marine Station.

On July 30 the President of the French Republic opened the new harbour works at Cherbourg, including the new railway station which had just been completed. The cost of the latter is approximately 69,000,000 frs., this figure including the costs of the reinforced concrete, foundations, roofing and walling, cranes and other mechanical equipment, roads, water mains, rail tracks and junctions connecting the new station to the existing lines. The cost of the reinforced concrete, foundations, and drainage was approximately 15,000,000 frs. A description of the harbour works during construction was given in this journal for December, 1929 and January, February, and March, 1930, the last article describing the new station buildings for which, as well as for the jetty, Messrs. Christiani & Nielsen were the contractors.

The station buildings cover an area of about 6¼ acres, and are founded on 924 pre-cast reinforced concrete piles 60 to 65 ft. long, driven to rock. The ground through which the piles had to penetrate is a filling consisting of a mixture of fine sand and gravel, below which is an old beach composed of a layer of sand and 18 ft. of clay resting on the rock. Through these strata it was impossible to drive the piles to the rock when a steam hammer was used, and the assistance of a water jet was found to be necessary. In a description of the work published in "Le Génie Civil" it is stated that the pump was driven by a 90-h.p. electric motor and had a discharge of 3,500 cb. ft. per hour. The pressure used was 140 lb. per square inch. With the aid of the water jet and a preliminary excavation 6 ft. deep at the position of the pile, the ground was rapidly removed under the point of the pile, and the latter descended to the clay bed under its own weight in a few minutes. A short period of driving with a hammer—on an average 75 blows—was then sufficient to enable the pile to penetrate the clay and reach the rock. In this way it was possible to drive ten piles in a day, the greater portion of the time being spent in moving the pile frame and in handling and lifting the piles. The operations of sinking and driving the pile were generally executed in about 15 minutes.

Bridge Types and Choice of Type.

By A. W. LEGAT, M.Inst.C.E.

INTRODUCTION—DIFFERENT TYPES—INVESTIGATION—NATURAL FACTORS — SUBFOUNDATIONS — DISTURBANCE — HEADROOM—LENGTH—ARTIFICIAL FACTORS—LOAD—WIDTH —CONTOUR—APPEARANCE—TYPICAL EXAMPLES — SPECIAL CONSIDERATIONS

Bridge Types Available.

EVERY site provides its own particular conditions affecting the choice of the type of bridge most suited to the situation. The number of different cases is so great that it is impossible to do more than sketch the general principles involved and to give some brief indication of the procedure the engineer should adopt in selecting the type of bridge for any particular site. There are roughly thirteen main types of bridge construction in reinforced concrete, most of which are again subject to variation in transverse arrangement in two or more of five different ways. In considering what type of bridge is best suited to any particular site therefore the engineer has a very wide choice before him, and the selection of the type best suited to the site conditions will call for careful consideration of every item of data he may be able to collect.

For most sites it will be found that several of the different types of construction will meet the local conditions, and the selection of these is not difficult. They may in fact be chosen by inspection of the types illustrated later, with little more than consideration on the lines of the accompanying notes. To choose from these the particular type which will meet the conditions with maximum efficiency, permanency, and economy is a much more complicated matter, and it will probably call for rough calculations and rough estimates to assist in the comparison of two or more of the possible schemes before the correct choice can be made. Whilst it is necessary to make investigation for each individual case encountered in practice, some assistance can be obtained by a consideration of the various factors which affect the choice, and it is the object of this article to define these factors and to suggest the general lines of investigation which may be followed.

The following notes relate to the thirteen different types of bridges referred to, and which are illustrated in *Figs.* 1 to 13 with the various transverse arrangements "A" to "E" shown in *Figs.* 14 to 18. Against each of the diagrammatic outlines is given the minimum span for which it would be usually considered; for some of the types are stated the maximum span constructed up to the date of writing, so far as the author has been able to ascertain, and the span range over which each type is commonly applied. It may be anticipated that the development of reinforced concrete, especially in regard to the use of higher

working stresses, will in future tend to increase the maximum span for each type.

Type No. 1 (*Fig.* 1) is a freely-supported slab or girder construction resting on abutments of any suitable form or material. This type is generally suited to spans up to say 35 ft. for a slab or a slab and beam construction, and to approximately 70 ft. if some form of parapet girder is used. It is simple to design and construct, and provided the bearings are properly arranged it offers a certain degree of " flexibility " in the event of bedding down or other disturbance of the substructure.

Type No. 2 (*Fig.* 2) consists of a series of continuous spans each of which may have a length up to say 70 ft. if parapet girders are used, but which would probably be limited to about 50 ft. in the case of girders below the slab and to 30 ft. if solid slab construction were adopted. In this type stresses set up by contraction and due to temperature changes must be taken into account. These may necessitate a cross division of the bridge by expansion joints at intervals if the total length is considerable. Care will also be required in the design of the bearings on the piers and abutments to avoid any considerable lateral stresses being transmitted to the piers.

Type No. 3 (*Fig.* 3) is an open frame construction in which the horizontal deck slab is made monolithic with the vertical abutment walls. This construction is suited to spans up to say 50 ft. but would not generally be found economical for spans exceeding about 30 ft. It is a type which might probably be adopted with advantage more frequently than has been done in the past.

Type No. 4 (*Fig.* 4) is a series of continuous spans in which the superstructure is not only continuous in itself but is also monolithic with the supporting abutments and piers. The span limits are similar to those of Type No. 3. This is a form of construction which should not be adopted where there is any doubt of the rigidity of the subfoundation, for it will be apparent that any bedding down of the piers or abutments relative to the other foundations would induce extremely high stresses. For ideal situations, as for example on rock foundations, the method would be economical.

Type No. 5 (*Fig.* 5) consists of continuous girder spans having varying moments of inertia along the span. As shown, this gives to the soffit of each span a shape similar to an arch, but where the girders are designed to project above the deck the structure may resemble a cantilever girder or a suspension bridge in outline. This is a type of construction suited to much longer spans than those previously mentioned, and it might be used with advantage in favourable circumstances up to a span of 150 ft. or even more. The superstructure might be made continuous with the piers and abutments or might be provided with free supports on the top of the piers and abutments. Generally speaking there is an advantage in providing joints, so avoiding somewhat doubtful variations of stress on the substructure and subfoundations.

Type No. 6 (*Fig.* 6) is suitable where the bridge is divided into several spans. Its main feature is the construction of alternate spans with projecting cantilevers the ends of which are used as supports for freely-supported spans constructed between them. This construction has been used for spans up to 200 ft. It has evidently greater " flexibility " than any of the continuous spans already mentioned and is therefore particularly well suited to sites liable to

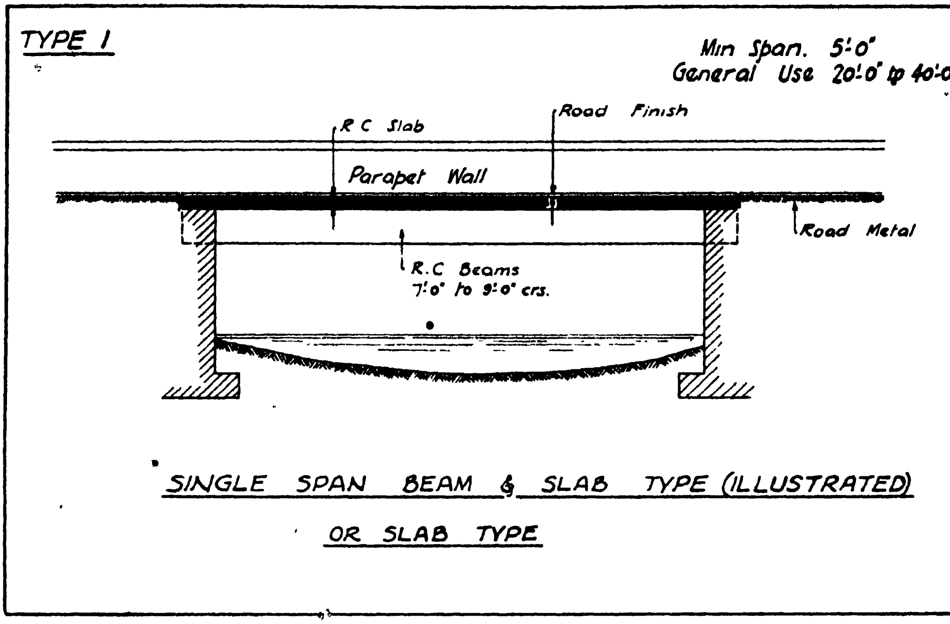

TYPE I
Min Span. 5'-0"
General Use 20'-0" to 40'-0"
R C Slab
Road Finish
Parapet Wall
Road Metal
R.C Beams
7'-0" to 9'-0" crs.
SINGLE SPAN BEAM & SLAB TYPE (ILLUSTRATED)
OR SLAB TYPE

Fig. 1.

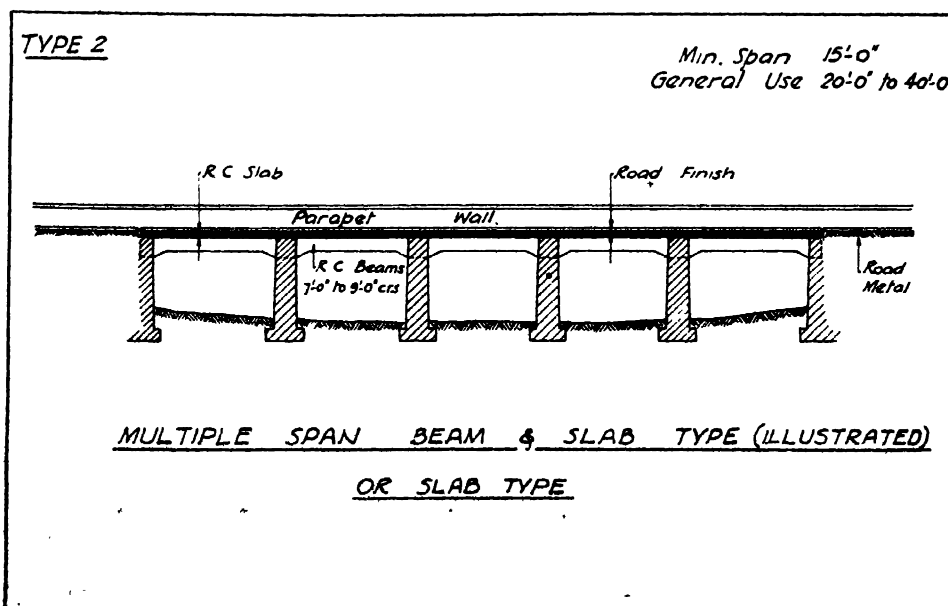

TYPE 2
Min. Span 15'-0"
General Use 20'-0" to 40'-0"
R C Slab
Road Finish
Parapet Wall.
R C Beams
7'-0" to 9'-0" crs
Road Metal
MULTIPLE SPAN BEAM & SLAB TYPE (ILLUSTRATED)
OR SLAB TYPE

Fig. 2.

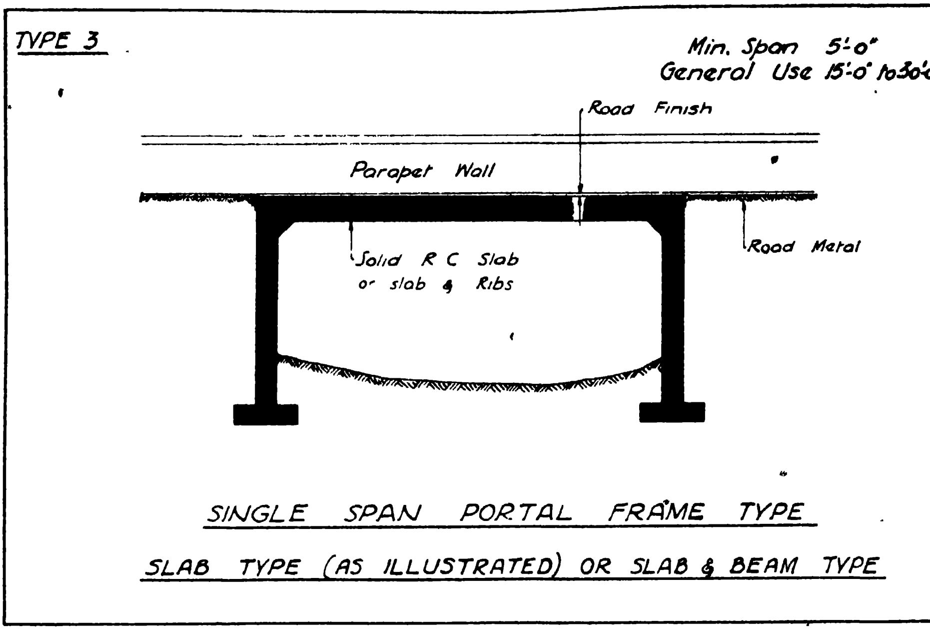

TYPE 3
Min. Span 5'-0"
General Use 15'-0" to 30'-0"
Road Finish
Parapet Wall
Road Metal
Solid R C Slab
or slab & Ribs
SINGLE SPAN PORTAL FRAME TYPE
SLAB TYPE (AS ILLUSTRATED) OR SLAB & BEAM TYPE

Fig. 3.

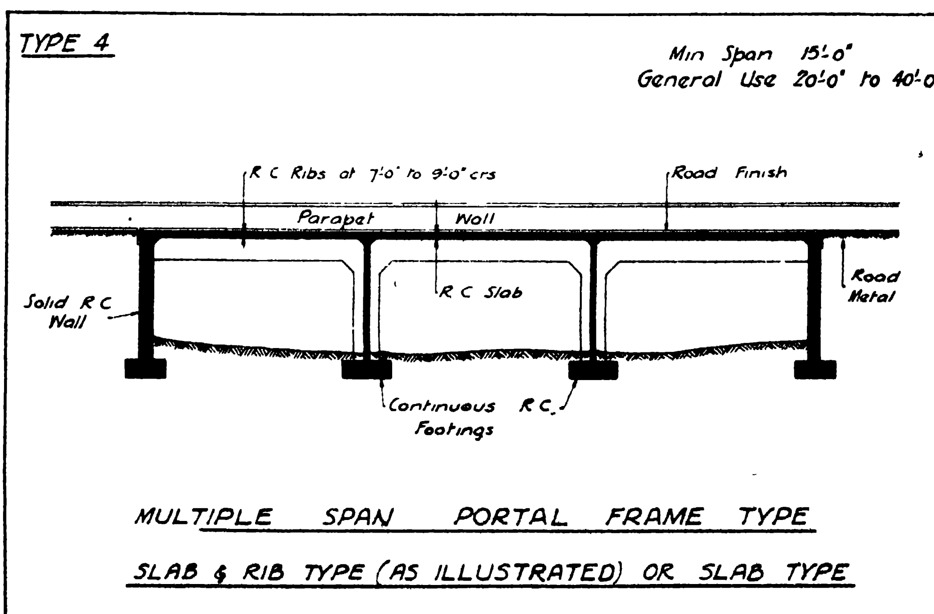

TYPE 4
Min Span 15'-0"
General Use 20'-0" to 40'-0"
R C Ribs at 7'-0" to 9'-0" crs
Road Finish
Parapet Wall
R C Slab
Road Metal
Solid R C Wall
Continuous R C Footings
MULTIPLE SPAN PORTAL FRAME TYPE
SLAB & RIB TYPE (AS ILLUSTRATED) OR SLAB TYPE

Fig. 4.

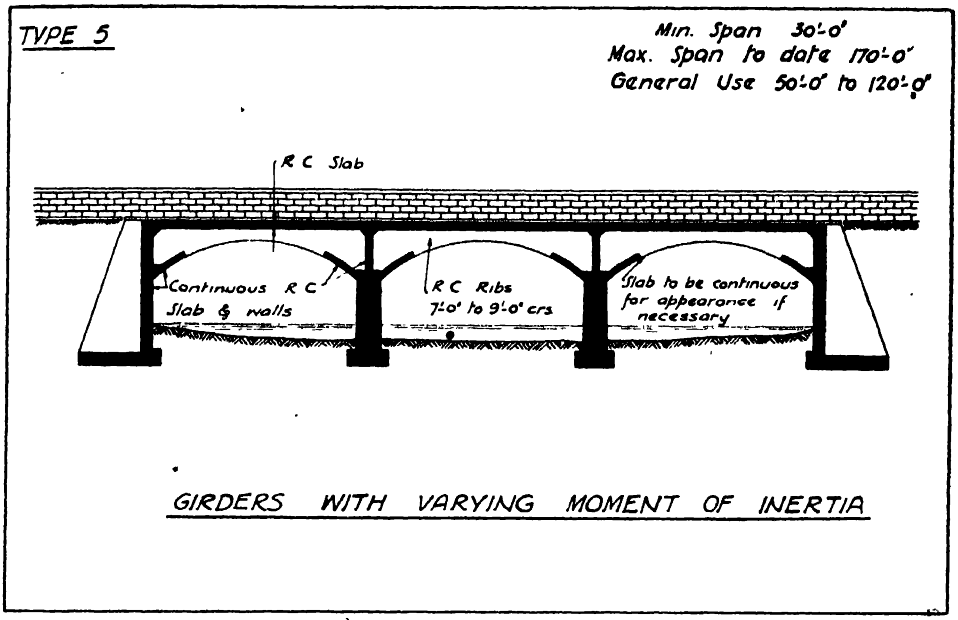

TYPE 5
Min. Span 30'-0"
Max. Span to date 170'-0"
General Use 50'-0" to 120'-0"
R C Slab
Continuous R C Slab & walls
R C Ribs 7'-0" to 9'-0" crs
Slab to be continuous for appearance if necessary
GIRDERS WITH VARYING MOMENT OF INERTIA

Fig. 5.

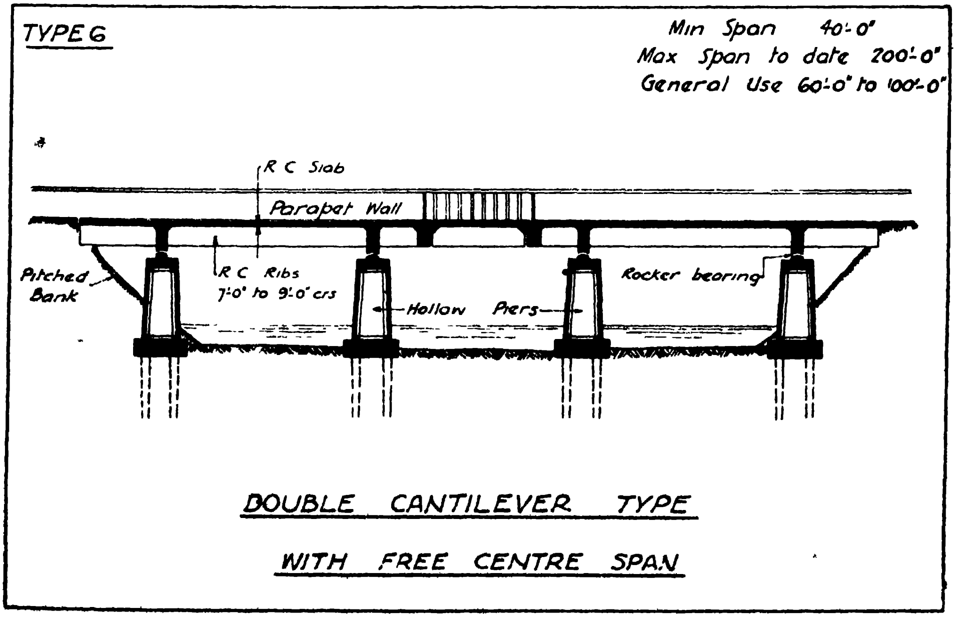

TYPE 6
Min Span 40'-0"
Max Span to date 200'-0"
General Use 60'-0" to 100'-0"
R C Slab
Parapet Wall
Pitched Bank
R C Ribs 7'-0" to 9'-0" crs
Hollow Piers
Rocker bearing
DOUBLE CANTILEVER TYPE
WITH FREE CENTRE SPAN

Fig. 6.

some bedding down or settlement of the supporting piers. For short spans up to say 40 ft. plain bearings may be used between the piers and the superstructure, but for longer spans a form of rocker or roller bearing which will permit of slight rotation due to loading, bedding down of the foundations, or other causes is an advantage.

Type No. 7 (*Fig.* 7) consists of a single long span between supporting piers, but with projecting cantilevers on the landward side of the piers at both ends forming counter-balances the effect of which is to reduce the positive moment in the centre span. This construction has been used for spans of up to 450 ft. between central supports. It is open to variation by the introduction of joints and a freely-supported span in the centre of the main span. This gives the construction greater "flexibility." It is a type open to considerable development and might with advantage be adopted in more cases than has been done in the past. Where the span is across a canal or river and towpaths are required on both sides the cantilevers projecting behind the main piers can be arranged to span over these paths.

Type No. 8 (*Fig.* 8) is a fixed arch, or arch made monolithic with the abutments or piers, and provided with closed spandrels, i.e. with side walls built up along both sides of the arch, the space between and over the arch barrel being filled with soil or other suitable material up to roadway level. It is suitable for use on spans up to 200 ft., or in special cases even more, but it is not often adopted for spans exceeding 120 to 150 ft. The reason for this is that the use of solid filling greatly increases the dead weight of the structure over and above that required if a construction similar to Type No. 9 is adopted.

Type No. 9 (*Fig.* 9) is a fixed arch in which the arch barrel or ribs are monolithic with the supporting abutments or piers. Instead of the solid filling noted in Type No. 8 the deck is carried by column, beam and slab construction, or cross-wall and slab construction. This type has been used successfully for the longest spans yet constructed in reinforced concrete, the most notable case to date being the Elorn Bridge at Brest having spans of approximately 600 ft.

Type No. 10 (*Fig.* 10) shows a three-hinged arch in reinforced concrete. It has been adopted with success for spans up to 200 ft. and is very suitable for situations where there is a possibility of slight movement in the subfoundation, as may possibly occur in certain types of soil or in areas subject to mining or other disturbance. Another advantage is the reduction of stresses due to contraction and temperature changes which become so important in monolithic structures. The hinges may be made entirely of metal, may be moulded in concrete, or may be formed in the reinforcement of the bridge.

Type No. 11 (*Fig.* 11) shows a two-hinged arch. This is a type which is not usually adopted. It has an advantage over the fixed arch in some cases, but as a general rule it will be found that if hinges are to be introduced then it is better to use three hinges, one at each springing and one at the crown, rather than to limit them to one at each springing as in this type. Here again the use of the hinges appreciably reduces the stresses due to contraction and changes of temperature.

Type No. 12 (*Fig.* 12) is the bowstring form of construction consisting of arch ribs constructed above the deck level of the bridge with horizontal ties

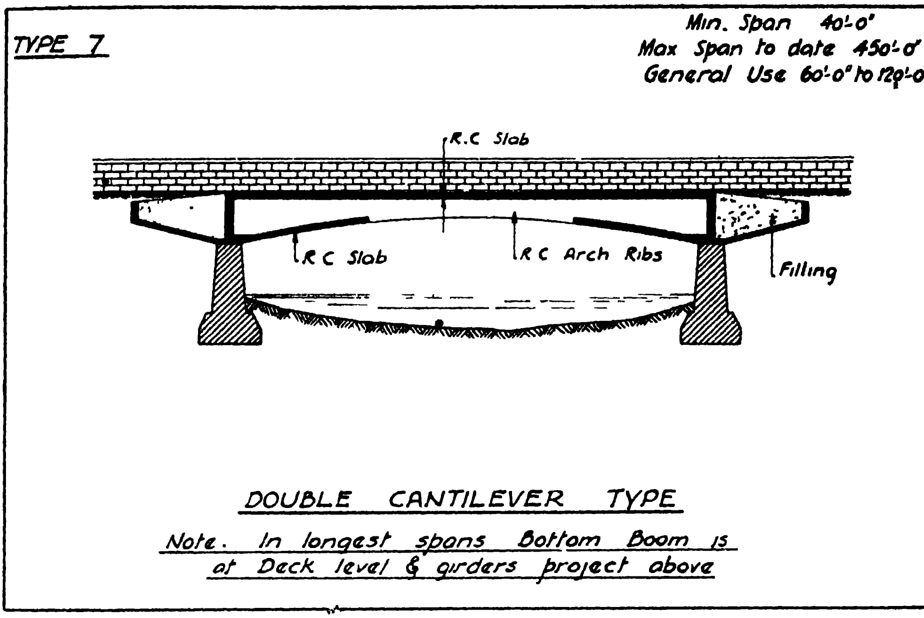

TYPE 7
Min. Span 40'-0"
Max Span to date 450'-0"
General Use 60'-0" to 120'-0"
R.C Slab
R C Slab
R C Arch Ribs
Filling
DOUBLE CANTILEVER TYPE
Note. In longest spans Bottom Boom is
at Deck level & girders project above

Fig. 7.

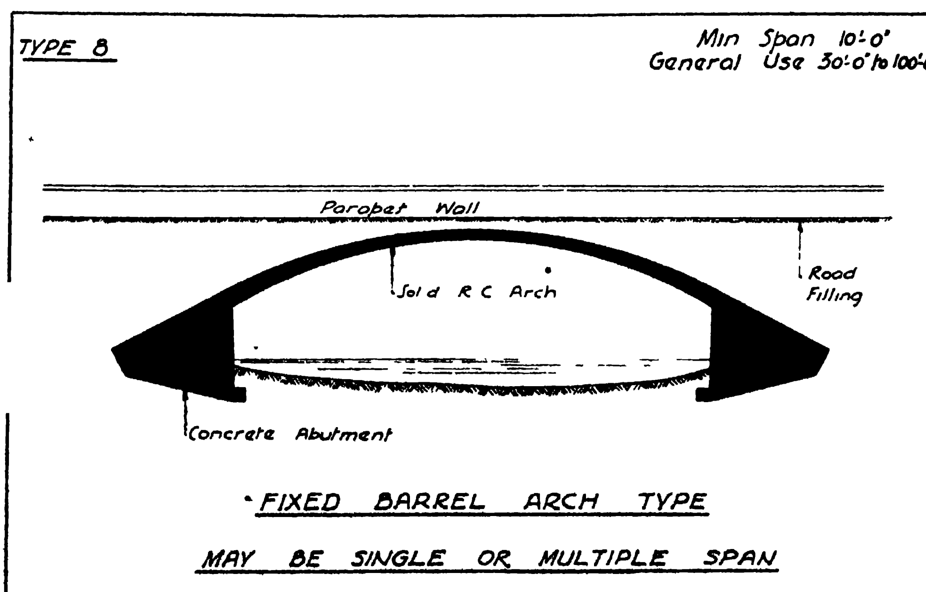

TYPE 8
Min Span 10'-0"
General Use 30'-0" to 100'-0"
Parapet Wall
Solid R C Arch
Road Filling
Concrete Abutment
FIXED BARREL ARCH TYPE
MAY BE SINGLE OR MULTIPLE SPAN

Fig. 8.

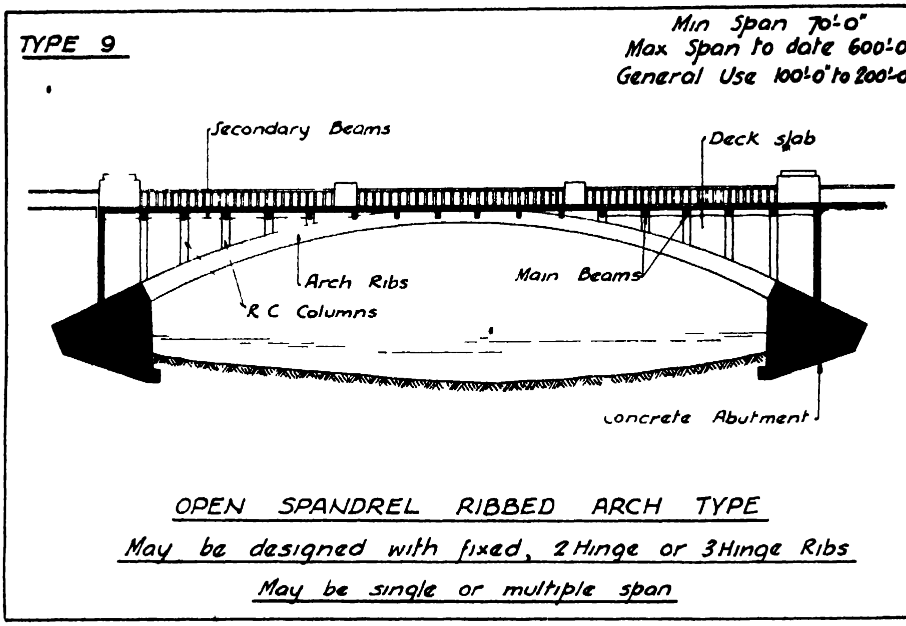

TYPE 9
Min Span 70'-0"
Max Span to date 600'-0"
General Use 100'-0" to 200'-0"
Secondary Beams
Deck slab
Arch Ribs
Main Beams
R C Columns
Concrete Abutment
OPEN SPANDREL RIBBED ARCH TYPE
May be designed with fixed, 2 Hinge or 3 Hinge Ribs
May be single or multiple span

Fig. 9.

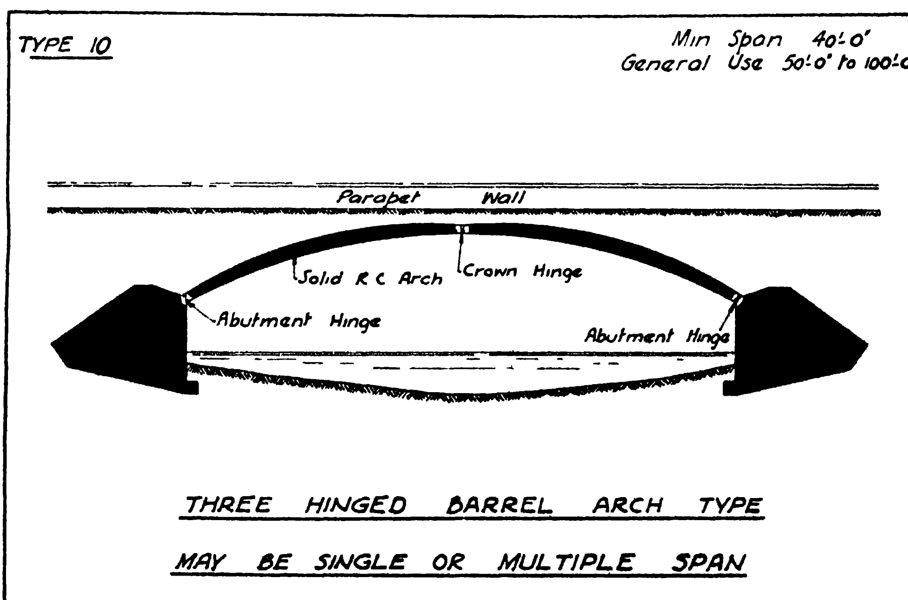

TYPE 10
Min Span 40'-0"
General Use 50'-0" to 100'-0"
Parapet Wall
Solid R C Arch
Crown Hinge
Abutment Hinge
Abutment Hinge
THREE HINGED BARREL ARCH TYPE
MAY BE SINGLE OR MULTIPLE SPAN

Fig. 10.

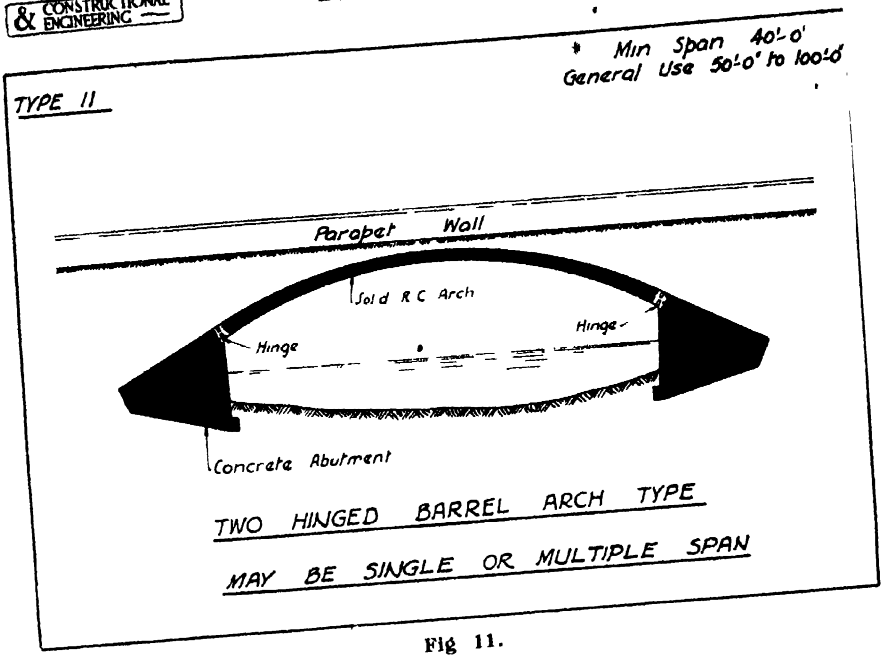

Min Span 40'-0"
General Use 50'-0" to 100'-0"
TYPE 11
Parapet Wall
Solid R C Arch
Hinge
Hinge
Concrete Abutment
TWO HINGED BARREL ARCH TYPE
MAY BE SINGLE OR MULTIPLE SPAN

Fig 11.

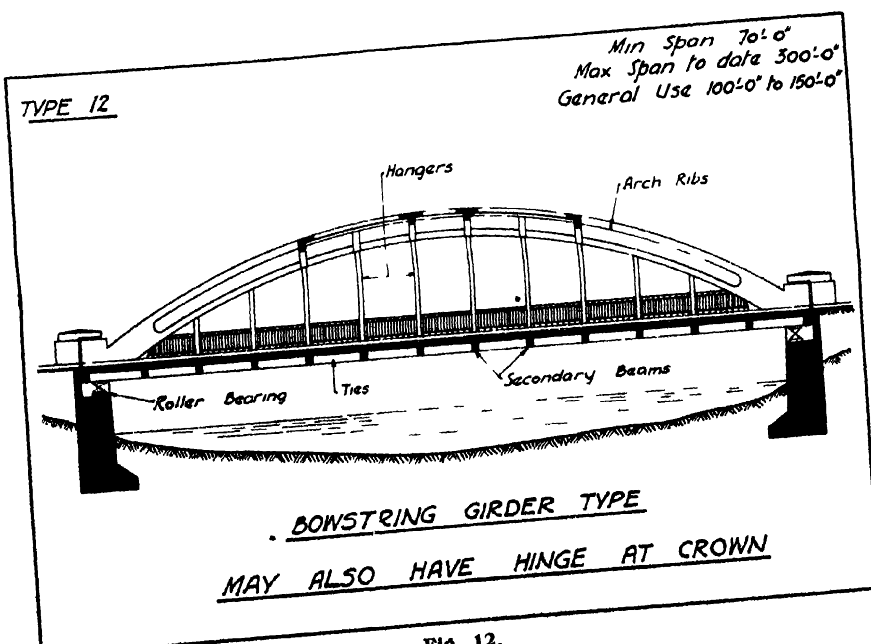

Min Span 70'-0"
Max Span to date 300'-0"
General Use 100'-0" to 150'-0"
TYPE 12
Hangers
Arch Ribs
Roller Bearing
Ties
Secondary Beams
BOWSTRING GIRDER TYPE
MAY ALSO HAVE HINGE AT CROWN

Fig. 12.

connecting the springings and resisting the horizontal thrust of the arches. These ties are therefore in direct tension. Usually they are suspended from the arch ring at intervals and form the side members of the deck construction. This form can be adapted to long spans, the longest noted to date being 300 ft. By some engineers the direct tension in hangers and horizontal ties is considered to be an objectionable feature tending to the development of hair cracks in the concrete which might result in deterioration of the reinforcement ; there is little direct evidence to support this criticism, however.

Type No. 13 (*Fig.* 13) is a variation of the bowstring construction in which the deck is placed at a level above the apparent springing of the arch. The deck is suspended by ties from the arch ring for the major portion of its length, but towards the end is supported from the top of the arch ring by means of columns or cross walls. This is a type which is not often seen and which the site conditions seldom make necessary.

Most of the thirteen types described are subject also to variation in the arrangement of the cross section of the bridge. These are indicated in *Figs.* 14 to 18 and are marked Types A, B, C, D, and E.

Type A (*Fig.* 14) consists simply of a slab of uniform thickness throughout the cross section. It is found in short-span portal-frame construction and in some parapet-girder bridges.

Type B (*Fig.* 15) consists of a deck slab supported by ribs or beams on the underside, and is possibly the most common form of deck slab construction in bridge work.

Type C (*Fig.* 16) consists of a deck slab supported by ribs or girders projecting above it along both sides. Its usual form is that of the parapet bridge.

Type D (*Fig.* 17) is similar to Type C except that the upstanding ribs are not so deep and do not form part of the parapet. It is not a very common form of cross section.

Type E (*Fig.* 18) is a cross section of a cellular type of bridge in which a soffit slab is provided along the bottom of the girders and the deck slab spans across the girders at the road level. Externally a bridge of this type has a solid appearance. Due to the reduction of dead weight which results it is a form suited to long spans, and will probably be used more in future than it has been in the past.

Types A, B, D and E can be used for any width of bridge, whilst Type C is suitable only for comparatively narrow bridges usually of the parapet girder type.

In addition to these general types and variations in reinforced concrete bridge construction one occasionally encounters what may appear to be unusual types, often simulating structural steel work in general arrangement and appearance. Actually these may almost invariably be included with one or other of the types illustrated. The great amount of intricate shuttering work which they involve makes them unsuitable for what are to-day considered normal spans. Due, however, to the saving of dead weight resulting from their open-work construction they appear to be more suited to very long span bridges, and it is mainly in this class of work that they have been used up to the present. For spans under 250 to 300 ft. it is doubtful whether they would prove economical.

Having before us the different types of bridge which may be adopted, there

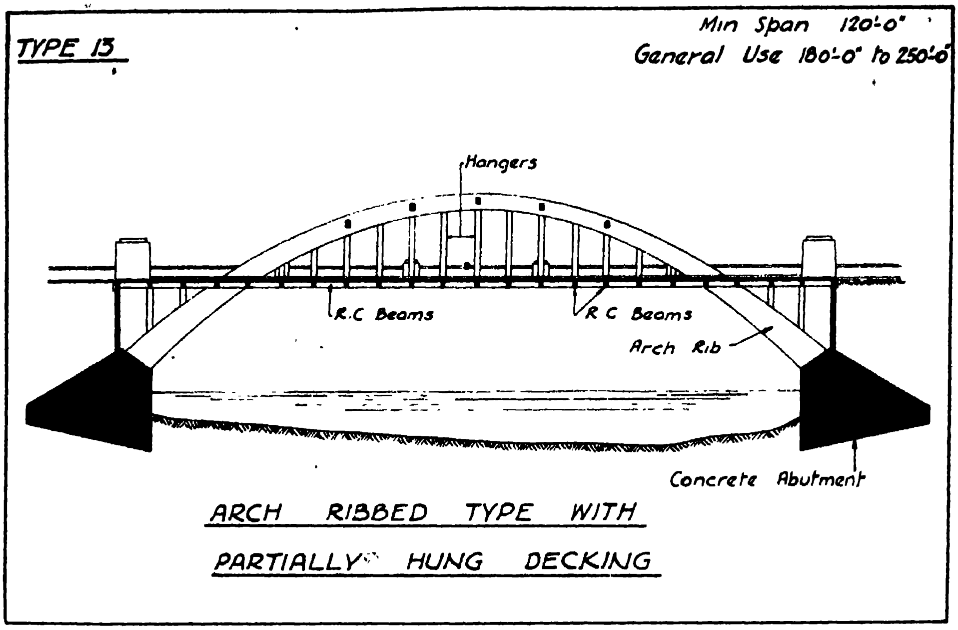

TYPE 13
Min Span 120'-0"
General Use 180'-0" to 250'-0"
Hangers
R.C Beams
R C Beams
Arch Rib
Concrete Abutment
ARCH RIBBED TYPE WITH
PARTIALLY HUNG DECKING

Fig. 13.

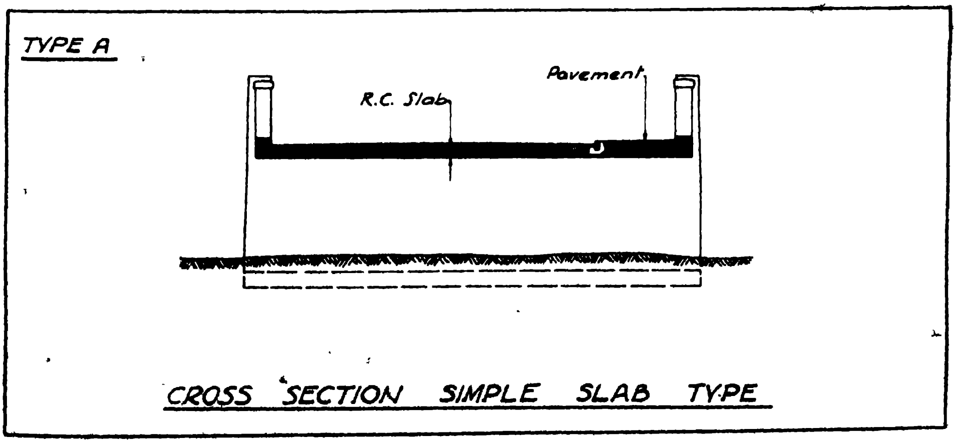

TYPE A
Pavement
R.C. Slab
CROSS SECTION SIMPLE SLAB TYPE

Fig. 14.

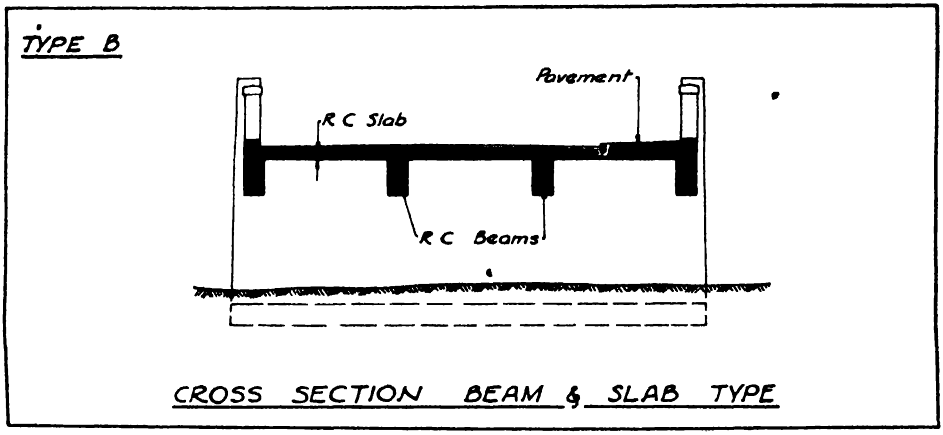

TYPE B
Pavement
R C Slab
R C Beams
CROSS SECTION BEAM & SLAB TYPE

Fig. 15.

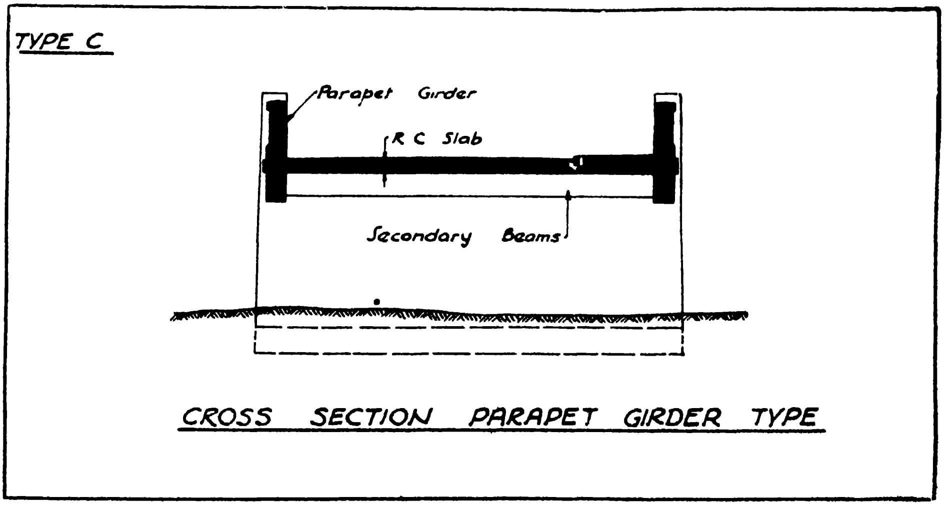

TYPE C
Parapet Girder
R C Slab
Secondary Beams
CROSS SECTION PARAPET GIRDER TYPE

Fig. 16.

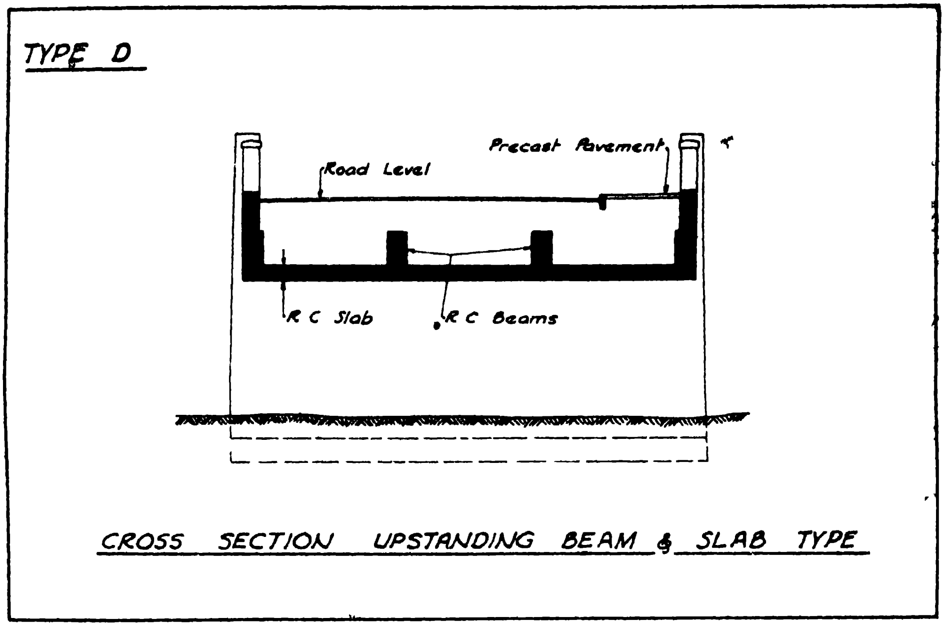

TYPE D
Precast Pavement
Road Level
R C Slab
R C Beams
CROSS SECTION UPSTANDING BEAM & SLAB TYPE

Fig. 17.

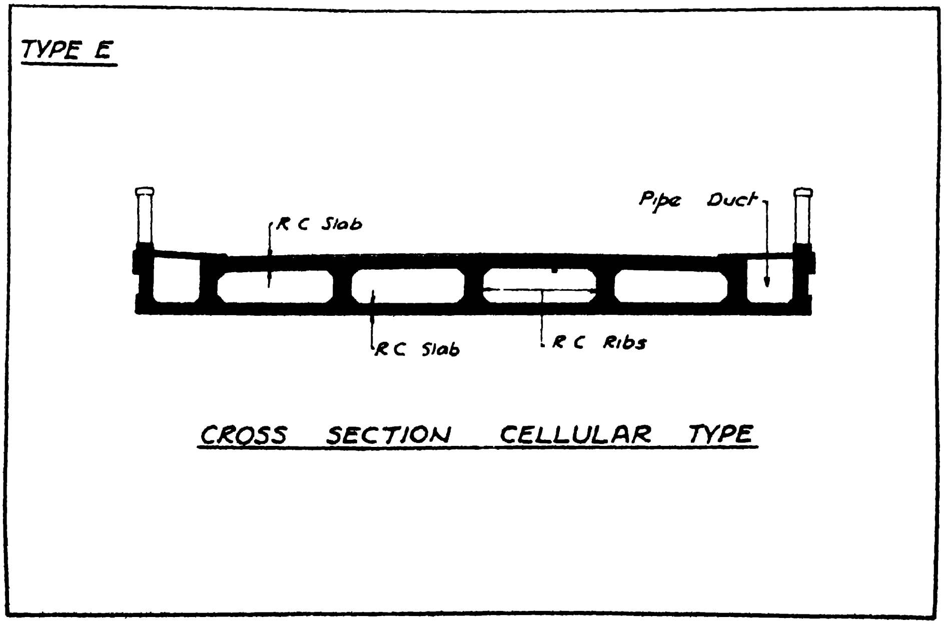

TYPE E
Pipe Duct
R C Slab
R C Slab
R C Ribs
CROSS SECTION CELLULAR TYPE

Fig. 18.

are three questions to which answers must be found before one can decide on the type of bridge most suited to any particular case.

(1) Which of the types can be adapted to the particular site in question?
(2) Which of these possible types will do the work most economically?
(3) Are there any disadvantages in the adoption of this most economical scheme? If so, one must then consider one of the schemes which is a little more costly than the cheapest to meet the requirements.

If we can find definite answers to these questions for any particular example then we shall be in a position to concentrate on the detail design of the chosen type knowing that the final design will have all the requirements of the ideal bridge and particularly those of efficiency, permanency, and economy.

In endeavouring to find an answer to these three questions, there are two sets of factors which call for consideration. The first, which may be termed the natural factors or factors dependent entirely on the site conditions, are:

(*a*) The subfoundation.
(*b*) The possibility of mining or other disturbance.
(*c*) Headroom necessary
(*d*) The length of the bridge.

There are also a number of what may be termed artificial factors, that is, factors decided by the necessities of the case These may be enumerated as:

(*a*) The load which the bridge is to carry.
(*b*) The width of the bridge.
(*c*) The contour
(*d*) The appearance.

Let us now consider these factors separately with a view to appreciating the extent and manner in which they affect the choice of bridge type.

Subfoundation.

It may be taken as a general rule that if the foundation work is comparatively costly it will be better to reduce the number of foundations as much as possible and in consequence to use comparatively long spans. If, on the other hand, the foundations are likely to be inexpensive it will be more economical to use short spans and a comparatively large number of foundations.

It has often been suggested that in multiple-span bridges the division of spans should be such that the total cost of the substructure equals the cost of the superstructure. Whilst there is some justification for this it should not be followed too rigidly in view of the many factors involved in the choice of type of bridge. These statements are, however, open to some qualification. Take for example the case where the ground is particularly poor and where in consequence it is necessary to spread the load from the new bridge over the whole area of ground covered by the bridge. This might conceivably occur on a site where it is impossible to obtain an absolutely rigid foundation at any reasonable depth which could be reached by piling, but where a comparatively low loading of the ground at shallow levels is permissible. Although the foundations in such a case would be comparatively expensive, it would obviously be an advantage to have several short spans rather than one long one in order to reduce the stresses induced in the raft system connecting the various piers and abutments.

From the general rule one is also led to the fact that, in most cases, if the foundations require to be carried to comparatively deep levels they will be costly and in consequence it will be better to use comparatively long spans and so reduce the number of foundations At the same time it should be noted that if a foundation is only obtainable at a very low level it will be better to have only vertical reactions, and in consequence for single spans the arch type of construction is not likely to be favoured in cases of this kind For multiple-span bridges, however, where the major portion of the horizontal thrust would be balanced over intermediate piers the objection would not apply so strongly. On the other hand, if the foundations are shallow they are likely to be comparatively cheap and short spans will be favoured Such shallow foundations will also be more suited to deal with inclined reactions, and an arch type of construction would therefore offer some advantage Where abutments or intermediate piers are to be constructed in a river or other water, dams are likely to be required These are temporary works carried out solely for the purpose of facilitating the construction of the permanent structure and are removed after the permanent structure is completed It is therefore desirable to limit the expenditure on such temporary works as much as possible, and this can best be done by reducing the number of foundations required This automatically guides one to a reduction in the number of spans, so that in such a case, other factors being equal comparatively long spans are favoured

The reason for the total cost increasing considerably with the increase in the number of dams is apparent, for as a general rule the longer side of the dam will be across the bridge and the short side in the direction of the span of the bridge For example, suppose a bridge pier is 50 ft long, say a little more than the total width of the bridge and has a width of say 12 ft , if two such piers are required the total length of dam required will be 248 ft If, however, four piers are used the width of pier to give the same intensity of loading on the ground will be in the neighbourhood of 6 ft and the total length of dam required for four piers would be 448 ft , or almost twice as much as in the first case In addition to the fact that the length of the dam increases almost directly as the number of supporting piers and abutments, the delay occasioned in constructing a number of such dams, timbering inside, pumping out, withdrawal of the dams after use, etc , is likely to be considerable and to increase appreciably the cost of the finished work

Table I gives the approximate weight per square foot which the superstructures of bridges of various spans are likely to produce, and if to these are added the design load per square foot calculated as equivalent to the live load, one can rapidly find approximately the total load on the foundations for any given span. From the information obtained during the investigation at the site the reasonable intensity of loading to be imposed on the ground at different levels will be known, and by equating the two one will be able to find roughly the width of foundation required for any subdivision of length If the total width of the foundation for the chosen span does not exceed 20 per cent to 25 per cent. of the span then it will usually be reasonable to spread the load from the pier. If, however, the spread required exceeds 25 per cent. of the span it is probable that it may be economical to go deeper with the foundations to a stiffer stratum or to use piles, or to deal with them in an even more elaborate

TABLE I.

APPROXIMATE WEIGHTS PER SQUARE FOOT OF GIRDER OR OPEN SPANDREL ARCH BRIDGES.

Span in ft	Approx dead load of bridge superstructure per square foot lb	Ministry of Transport approximate equivalent live load per square foot lb.
20	160	490
30	180	400
50	215	320
70	250	290
100	310	260
150	400	230
200	490	210

[*Note :* Add the dead and live load figures given to the approximate weight of the foundations per square foot of bridge This total multiplied by half the bridge area equals the load on one foundation]

EXAMPLE.

The use of the above table may best be shown by an example.

Suppose one is considering the adoption of a 100-ft. span girder bridge. The approximate width of foundations required may be determined as follows:

	lb.	
Dead load	= 310	from Table.
Live ,,	= 260	
Foundations, say $\frac{2}{3}$ weight of superstructure	= 200	
	770	

Safe intensity of pressure from foundations = 2 tons per square foot

$$\therefore \text{ width of one foundation } = \frac{50 \text{ ft.} \times 770}{2 \times 2240} = 8{\cdot}6 \text{ square feet.}$$

In the case of an arch bridge, the values of the live load and the weight of the superstructure could be taken from this Table, and the approximate horizontal thrust obtained from the formulae $H = \frac{Wl^2}{8r}$, where r is the rise of the arch. The size and shape of the foundations are then best determined graphically, being largely dependent on the height of the arch springing from the natural subfoundation level.

way. Having made rough calculations on this basis, the engineer will find it necessary to prepare rough comparative costs of foundations necessitated by different spans or types to guide him to an answer to the question " What type and span of bridge is to be adopted ? "

Subfoundations Liable to Disturbance.

In mining areas or in areas where brine is pumped from the ground there is considerable possibility of settlement taking place in the ground at great depths below the bridge foundations. It should first be appreciated that such movements are really miniature earthquakes, and it is quite impossible to prevent this type of disturbance from affecting the bridge structure no matter of what material or to what design it may be built. No amount of spread or increase in the depth of the foundations (short of carrying the foundations down to a depth below the mine workings) will prevent ultimate settlement taking place where the whole of the ground sinks for great depths. In consequence it is necessary for the engineer to keep in mind that the bridge may settle and that it may not settle uniformly. In such cases he should adopt a type which will allow for considerable relative variations in settlement, of say 6 in. upwards, of the abutments and various piers, dependent on the knowledge of the settlement already experienced in the neighbourhood, and the design must be such that this settlement may take place without affecting the strength or permanency of the structure. A factor of this kind will have a considerable effect on the choice of type of bridge to be built, and will always lead away from monolithic construction towards one of the " flexible " types having the maximum number of hinged joints.

In this connection it should be noted also that in such cases there is a slight possibility of greater settlement at one end of an individual pier or abutment than at the other. In other words, it does not necessarily follow that a pier will settle uniformly over its whole area ; it may in fact tilt across the bridge. The only way of providing for this contingency is to design and construct the pier as stiffly as possible along its length so that it will be capable of standing stresses induced by such unequal settlement, and by increasing the intensity of load on the part of the subfoundation which does not settle will tend to crush this and to keep itself as nearly as possible on an even keel. Reinforced concrete is particularly well suited to the construction in such cases. In steel bridges arrangements are sometimes made for the jacking up of the girders on the top of the abutments and piers in the event of non-uniform settlement There is no reason why a similar arrangement should not be adopted in reinforced concrete construction. The dead weight is generally greater, but usually it would not be found either difficult or unduly expensive to provide facilities for jacking up the reinforced concrete superstructure. Unfortunately there has been up to the present a tendency to decry the use of reinforced concrete for cases of this kind, but it is anticipated that in future there will be a slowly increasing adoption of reinforced concrete for bridges of this type. In such cases Types Nos. 6, 10, and 12 are suitable for consideration ; the remaining types should not be adopted.

Another very important factor in deciding the type of bridge to be used is the maximum construction depth available. The level of the soffit of a bridge

is usually governed either by the height of flood water or by any navigational use to which the river or stream may be placed. On the upper side the level will be governed by the contour of the roadway over the bridge, and the difference between the two will give the construction depth available. If this construction depth is large then it will in itself have no effect on the type of bridge to be used, but if it is restricted and it becomes less than a twentieth part of the span then the girder type of construction (Nos. 1, 2, 3, and 4, and possibly also No. 6) is not likely to be suited to the case. Instead it will be found that Type No. 12, the bowstring girder, will give the maximum of construction depth owing to the fact that the actual carrying members of the bridge project well above the roadway. Types Nos. 5, 7, 11, and 13 will probably be the next most suited for a case of this kind. As regards the cross section of the bridge, Type A would probably give the minimum of constructional depth, followed closely by Types C and D. The construction depths necessary for a roadway say 20 ft. in width, using Type No. 12 bowstring girder, will probably be from 18 in. to 24 in. and will be quite independent of the bridge span and wholly dependent on the width between the bowstrings and also on the load to be carried. The arch construction (Types Nos. 8, 9, and 11) and the girders having varying moments of inertia (Type No. 5) will probably necessitate a constructional depth in the centre from a thirtieth or a fortieth part of the total span of the bridge, depending to some degree on the constructional depth available at the springing or alternatively on the rise of the arch.

Length of Bridge.

The total length of the bridge is another factor affecting the type to be adopted, and the length itself depends almost entirely on the economic point at which the change should be made from suspended to solid construction. It resolves itself into a comparison of costs of suspended approaches and of solid approaches consisting of approach walls and the necessary filling. A table similar to *Table* II based on rates applicable to the site will be of assistance. This gives the approximate cost of retaining walls of various heights on normal ground together with the cost of filling per foot of length for varying heights and for approaches having a total width of 20, 30 and 40 ft. The cost of the filling is based on a rate of 3*s*. 6*d*. per cubic yard in place. If we take the cost of construction of suspended approaches at say £10 per square yard and compare this with the cost of the solid approach construction we can quickly find a suitable point at which to change from solid to suspended work. As a general rule this will be found to occur where the bank in the approach increases beyond 20 ft. in height. It should be noted that whilst the prices given are suitable for normal conditions and rates at the present day they will vary for different districts as the prices of the various materials involved vary from time to time. In consequence it will be necessary for the engineer when he has an important bridge structure under consideration to make up approximate prices based on the local rates ruling at the time on the same lines as those shown for purposes of comparison.

There are numerous factors which may considerably influence this decision. For example, in some cases the materials excavated may be gravel suitable for

use in concrete, and even if not suitable for reinforced concrete it will be reasonably suitable for mass concrete. In a case of this kind the retaining walls to the approaches would be more economical to construct than in normal cases, and in consequence it would probably be an advantage to carry out the solid approach work farther and higher than in a normal case.

A similar effect would result if filling were particularly cheap or if it was necessary to find a suitable tip for the excavations for the bridge foundations. Here it would be advisable to arrange the approach works so that they would absorb the whole of the surplus excavation available.

Another case might occur where it was wished to make some improvement to a corner of the roadway at a site not far distant from that of the bridge and where there might not be available a suitable tip for any surplus soil. In such a case use might be made of the approaches of the new bridge to deposit the soil from such an improvement.

The type of subfoundation will also have to be taken into account. In cases where suspended work necessitates deep and costly foundations, while solid construction can be founded cheaply at shallow levels, then the latter will obviously offer an advantage and will in consequence be adopted as far as possible.

It will also be noted from consideration of *Table* II that the narrower the bridge the longer will be the total span, from solely economical reasons due to the fact that the approach work will always call for two supporting walls which are much more expensive than the intervening filling and more closely approach the unit cost of suspended work. Usually, therefore, the narrower the bridge the longer the suspended work and the shorter the solid approach; the wider the bridge, the shorter the suspended work and the longer the solid approach work.

A further factor which might have some effect on the position at which one changes from solid to suspended work is the cost of the land on which the bridge is to be built. If this was particularly high it would be desirable to limit the width taken up by the banking of the approaches as much as possible and in consequence this banking would probably be retained between walls for the whole of its length.

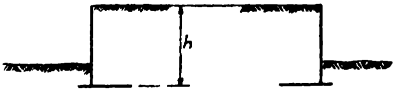
These various factors affecting the point where the change should be made from solid to suspended work have been briefly mentioned, but their number might be considerably extended; in fact the data collected at the site of any proposed bridge will almost invariably contain some novel factor affecting this choice. Those mentioned will, however, provide some guidance to the line of thought necessary when considering this matter.

In addition to consideration of the points at which the change from suspended to solid work should be made, one must also consider the question, " When should one change from main spans to suspended approach spans ? " This is a factor almost invariably settled by the contour of the site, but in principle it should be noted that down to comparatively short spans the greater the number of supports the lower the cost of the suspended work per unit of area on normal foundations.

Another factor which will enter into the question is that of the width of the river to be spanned, and as has already been noted it is essential that the

TABLE II.

APPROXIMATE COSTS OF SOLID CONSTRUCTION FOR BRIDGE APPROACHES.



Height in ft.	Cost per lin. ft.		Cost per lin. ft. of filling at 3*s*. 6*d*. per cub. yd.			Note.
	1 wall	2 walls	Bridge 20 ft. wide	Bridge 30 ft. wide	Bridge 40 ft. wide	
8	22*s*.	44*s*.	62*s*.	93*s*.	124*s*.	Add cost of 2 walls and filling between, and where this is less per square yard than the cost of suspended work, or say approximately £10, then solid construction is probably preferable.
10	32*s*. 6*d*.	65*s*.	78*s*.	117*s*.	156*s*.	
12	45*s*. 6*d*	91*s*.	93*s* 6*d*.	140*s*.	187*s*.	
15	70*s*. 6*d*.	141*s*.	117*s*.	175*s*. 6*d*	234*s*.	
18	101*s*.	202*s*.	140*s*.	210*s*.	280*s*.	
20	127*s*.	254*s*.	156*s*.	234*s*.	312*s*.	
25	203*s*.	406*s*.	194*s*. 6*d*.	292*s*.	389*s*.	
30	304*s*.	608*s*.	233*s* 6*d*	350*s*.	467*s*.	

EXAMPLE.

A bridge approach is to be 40 ft. wide and 25 ft. above foundation level. Will solid or suspended construction be more economical ?

From the Table :

Cost of two walls = £30 8*s*. 0*d*.
,, filling = 23 7*s*. 0*d*.

£53 15*s*. 0*d*.

$$\text{Cost of solid construction per square yard} = \frac{£53\ 15s.\ 0d. \times 9}{40}$$

$$= £12 \text{ approx.}$$

Suspended construction (cost £10 per square yard approximately) would probably be the cheaper.

full area of waterway existing before a bridge is built shall be preserved in the design of the new structure.

Loads, Width and Contour.

Next we come to artificial factors affecting the choice of type. In the first place we have the question of load. Where this is heavy it will tend towards the adoption of comparatively short spans, whilst if the design load is light it will tend towards the adoption of longer spans. The reason for this is that the heavier the load the heavier the construction required to carry these loads and the more expensive in consequence does the suspended work become.

The next question is that of width of bridge. This has not a very great influence on the type of construction to be chosen except in cases where the bridge is so small that parapet girders may be adopted. These will provide an economical form of construction, but they are not generally liked owing to the possibility of vehicles colliding with them, and in damaging the parapet at the same time damaging the supporting power of the structure. This is a factor which does not occur if the parapet is not part of the supporting structure of the bridge as is the case in every other type.

The next factor is the contour of the bridge, and this can be adapted to any type except that of the bowstring, which as a general rule is made horizontal or with only a very slight camber along the length of the bridge rising towards the centre.

Appearance.

Finally there is the question of appearance. Where the bridge is to show a concrete finish any of the types may be adopted, but where it is wished to add a veneer of stone or other material then only solid spandrel construction is suitable. This can be obtained in Types Nos. 1, 2, 3, 4, 5, 6, 7, 8, 10 and 11.

A decision on the subdivision of the total length of a bridge is also frequently affected by the appearance. Unless there is special reason to the contrary, multiple-span bridges should have an odd number of spans; an even number never looks well unless there is a special reason obvious to the layman when the bridge has been constructed.

In certain situations some of the types may appear incongruous and unsuitable, and this is a factor which should be given careful thought for it is unwise for the engineer when designing a new structure to offend the æsthetic taste of the general public. He must remember, however, that the purpose of a bridge is of greater importance than its appearance, and it is seldom that the type of bridge to be adopted will be governed by its elevation. Instead, when an architect is not employed, it will be a problem for the engineer to make the elevation of the most suitable type of bridge fit in as well as possible with its surroundings. We know of a case where a bowstring girder has been erected in a wild moorland situation, and which in consequence has been severely criticised on the score of its unsuitable elevation. In this case the bridge crosses a stream traversing a wide valley, the stream being liable to sudden and high floods. To have raised the roadway in the neighbourhood of the bridge in order to permit of an arch or girder type of construction would not only have been expensive but might also have been dangerous due to the possibility of floods overflowing the

river bank and causing damage to the new embankment on both sides. In consequence a bowstring girder was adopted requiring the minimum constructional depth and so providing the maximum headroom for the flow of water. This is a perfectly sound engineering decision. It is admitted that the appearance of the bridge is not quite suited to the countryside. Possibly a roughly bush-hammered stone, concrete, or masonry-faced arch would have been more fitting, but this is a case in point where although the bowstring is unsuitable from the æsthetic point of view it is nevertheless the only sound engineering solution of the problem.

Examples.

Having suggested in brief outline the various questions which arise in selecting a type of bridge, it is proposed to apply the conclusions to a number of typical cases where bridge construction might be required. It must be understood that every bridge type requires separate consideration of all the factors mentioned before a final decision can be arrived at, and the three examples given are merely a guide to the method of analysis and are not the analysis itself for any particular actual example.

EXAMPLE 1.—A deep and narrow rocky ravine, with rock outcropping at both sides and everywhere found at shallow depths below the surface soil.

This is obviously a case where it would be expensive to bring a foundation up from the bottom of the ravine to a comparatively great height and consequently at considerable cost. The case would therefore lead one toward the adoption of a single-span bridge. If the span were up to approximately 30 ft., Type No. 1 might be adopted. The rocky side of the ravine could, however, provide excellent resistance to the thrust of an arch, and in consequence Type No. 8 would also call for consideration due to the fact that there would be no likelihood of settlement in the rocky sides of the ravine. The fixed type arch No. 8 would here be preferable to the hinged type Nos. 10 or 11. As there are no practical limits to the construction depths this factor does not affect the decision. The choice would probably rest between Types Nos. 1 and 8, and a comparison of cost and appearance would finally determine which of these types would be used.

EXAMPLE 2.- A rocky valley, comparatively shallow and narrow, with the rock to be found everywhere at comparatively shallow depths below the surface. It is probable that in such a case the stream in the valley would be liable to considerable flooding.

In this case the foundations would evidently be inexpensive, and in consequence it would be reasonable to use comparatively short spans. There is no possibility of mining or other disturbance, and in consequence there is no necessity to make the bridge particularly "flexible." Owing to the probability of considerable flooding it would be desirable to provide the maximum waterway and also to give plenty of length to the bridge in order not to affect either the flow of the river in flood or the distribution of flood water when the river overflows its banks. In consequence, if the span does not exceed 30 ft. Type No. 1A (*Figs.* 1 and 14) would require consideration. If the span is greater, Types Nos. 1B or 1C could be adopted for spans up to say 70 ft., although in view of the comparatively inexpensive foundations it would probably be found more economical

to subdivide the total length into two or more spans. If the total width were 70 ft. or more, and if the construction depth available was very little indeed, and if a single span offered some decided advantage, then a bowstring type of construction might call for consideration. The possibilities here would probably be narrowed considerably by the detail data for any particular case, and as in Example 1 would probably resolve itself into the comparison of cost and appearance of, say, two of the types mentioned.

EXAMPLE 3.—A wide deep valley with a stream to be crossed by the bridge, the ground being of normal type.

In this case the foundations and substructure would be likely to prove expensive due to their considerable height, and in consequence it would be desirable to limit their number as much as possible. Further, due to their height, as previously noted it would be better to produce only vertical reactions. This rules out arch construction unless the arches are made with very considerable rise and spring from a level close to the ground. No mention is made of mining or other disturbance, and in consequence this factor will not affect the choice. Due to the depth of the valley it is obvious that the question of construction depth and headroom under the bridge would not affect the decision. The total length of the bridge would in this case probably require careful consideration, as also the subdivision of this total length into suitable spans. It is probable that Types Nos. 5 or 6 would prove most suitable for this case, probably adopting cellular construction for the piers.

In these considerations no mention has been made of skew spans, and it should be noted that these have some bearing on the decision arrived at in regard to the type of bridge to be adopted as they invariably create complications in the design although little if any in the actual construction. Types Nos. 1 and 2 are more suitable than Types Nos. 3 and 4 where spans of any considerable skew are under consideration. The reason is that in a skew span there is always a doubt as to the direction in which the load will span, and there is a probability that it will tend to carry across the shortest span, that is, normal to the line of the supports as shown on the accompanying *Fig.* 19. In such a case one would be left with a state of affairs not amenable to strict calculation in the corners of the bridge span. In these positions it is possible that fairly high concentration of stresses will occur on which it is impossible to place a calculated value. Type No. 5, with accurate calculation, is quite easily adaptable to skew spans. The same applies to Type No. 6 except that one is here faced with a somewhat peculiar state of affairs in regard to the hinges. These will not usually be normal to the centre line of the bridge, and in consequence their movement, due to various factors of load, contraction, temperature changes, etc., is somewhat complicated. For spans up to say 60 ft. this will not be a serious matter, but for spans of over say 100 ft. it would require careful consideration. It should be remembered that in cases of this kind, and in fact in every skew bridge, the reinforcement of the acute angle both of the support and of the suspended work should receive special consideration, and special reinforcement should be supplied to meet the stresses which it is evident are likely to develop and on which it is impossible to place any accurate calculation.

It will be appreciated that the choice of bridge type depends to some degree on experience, but even more on a careful study of the site conditions. At

first it will prove somewhat laborious to go into the various considerations and to produce rough designs and rough estimates of the various schemes considered. In time the choice of type becomes much more straightforward as one obtains guidance from the rough schemes and the final schemes adopted for earlier work. In nearly every case there is a particular type which will give the most satis-

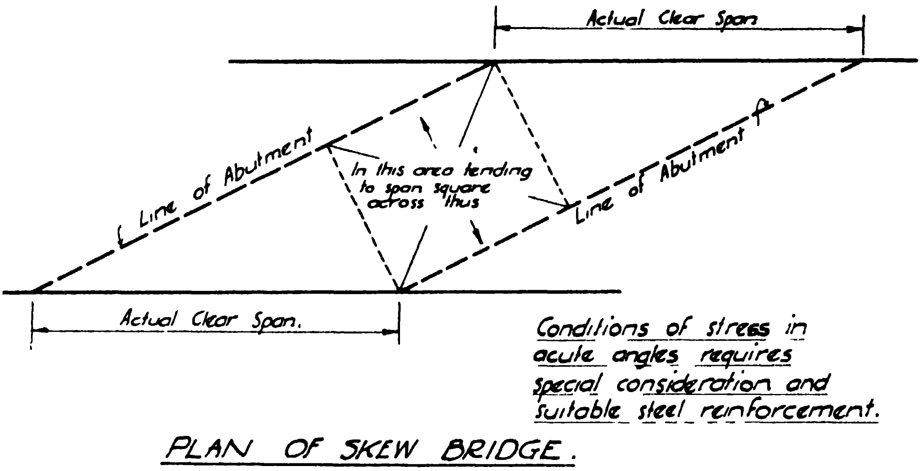


Fig. 19.

factory finished structure for the purpose for which it is built and which is also the most economical to meet the requirements of the case.

In conclusion it should perhaps again be emphasised that this article provides a general indication of the line of thought required in selecting the type of bridge for any site, but that the decision for any particular case depends almost entirely on the site or on the natural and artificial conditions which have been noted.

The Ryburn Dam.

On September 7 the Wakefield Corporation's mass concrete dam in the Ryburn valley was completed at a cost of about £245,000 The new dam, which was constructed by direct labour to the design and under the supervision of Mr Clemesha Smith, M Inst C E, the Waterworks Engineer to the Wakefield Corporation, retains water to a maximum depth of 100 ft and is curved on plan to a radius of 700 ft The maximum height from foundation level to the roadway across the crest is 121 ft, and the storage capacity is 220,000,000 gallons At foundation level the maximum width of the base is 84 ft.

The whole of the concrete has been placed by gravity through 10-in. sheet iron tubes, and the depth of the cut-off trench has been reduced by extensive use of cementation.

Illustrated articles describing the design and construction of the dam were published in this journal in July and August, 1930.

Promenade Development at Wallasey.

IN the year 1927 Parliamentary sanction was obtained for the extension of New Brighton promenade to a point 300 yd west of Harrison Drive, at an estimated cost of £900,000 The work now in hand forms the first portion of the scheme designed by Mr I St G Wilkinson M Inst C E, the Borough Engineer and consists of the extension of the promenade to the Red Noses, a distance of about ¾ mile, and the provision of a marine lake, open-air swimming bath, and public gardens, at a cost of about £500 000 The contractors are Messrs Edmund Nuttall, Sons & Co, Ltd

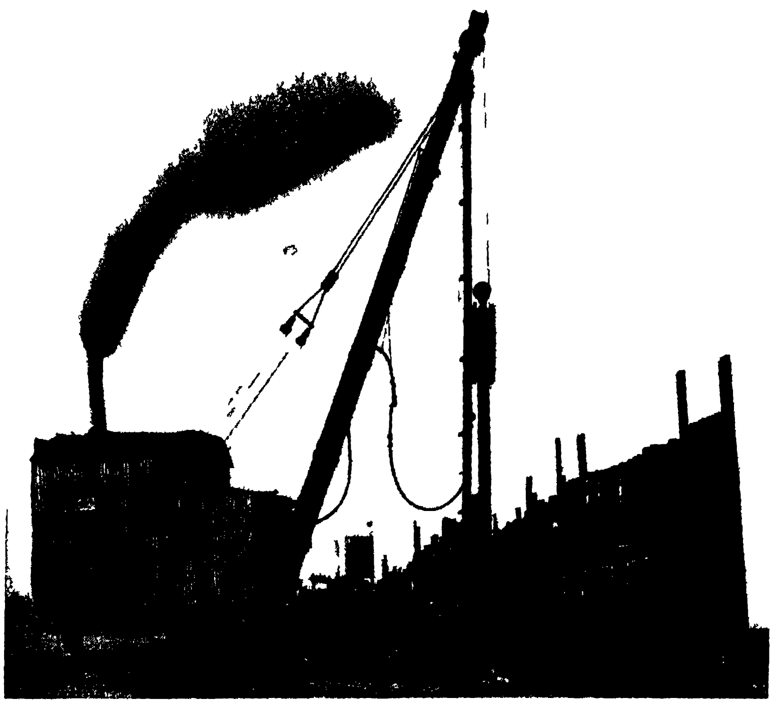


Fig. 1.

An area of 40 acres is enclosed and reclaimed from the foreshore by a new wall containing about 50,000 cb yd of mass concrete Ten acres are used as a boating lake 4 ft 6 in deep, which is filled by spring tides lapping over the lake wall Valves are also provided to enable the lake to be filled on smaller tides The remainder of the enclosed space is being laid out as promenades, gardens, a motor park, and an open air swimming bath

The Sea Wall.

Work on the new sea wall was commenced in April, 1931 and this and the marine lake are now completed The wall is constructed of mass concrete 16 ft thick at the base, and the total height from foundation to coping level is 38 ft 8 in at the deepest point, one half of this height being below shore level The foundations are partly on clay and partly on the red Triassic sandstone In each case the wall was constructed inside a cofferdam of Larssen No 2 steel sheet piling driven below foundation level and sufficiently high to exclude the tides At

the deepest section the piles were 40 ft long. A No 7 McKiernan-Terry steam hammer, slung from the jib of a crane mounted on caterpillar track (*Fig.* 1), was used for driving the piles, two piles being driven simultaneously. Compressed-air jetting was used to assist in driving the sheet piles, and proved a very successful method of increasing the rate of penetration. The pressure of compressed air used in driving the steel sheet piling was 90 lb per square inch. A 1½-in. diameter pipe and a ⅜-in nozzle were used. The compressed air acting in conjunction with the water in the sand through which the piles were being driven was found to reduce the average time required for driving a pile from 35 minutes to 25 minutes.

Where the wall foundations were on rock the steel sheet piling was withdrawn, but where the foundation was clay the lower portions of the piles were left in position and 14-in. square precast reinforced concrete piles were driven at 10-ft. longitudinal centres under the toe of the wall to obtain a greater degree of stability. The reinforced concrete piles were driven to a set of 1 in. for 10 blows of a 50-cwt. drop hammer falling 3 ft. The number of piles required was 370, having a total length of 5,760 ft.

On the land side the wall thickness decreases by stepping back 6 in. at vertical intervals of 2 ft., while the sea face is stepped as shown in *Fig.* 2, the profile being designed to break the impact of the waves, return the water to the sea, and keep the promenade free from water other than spray blown over by the wind

Material excavated from the cofferdam was tipped behind the wall. Additional filling was obtained from excavation of the sandhills by a Ruston-Bucyrus navvy on caterpillar track. The total volume of filling required for bringing the promenades to the required level was approximately 280,000 cb. yd.

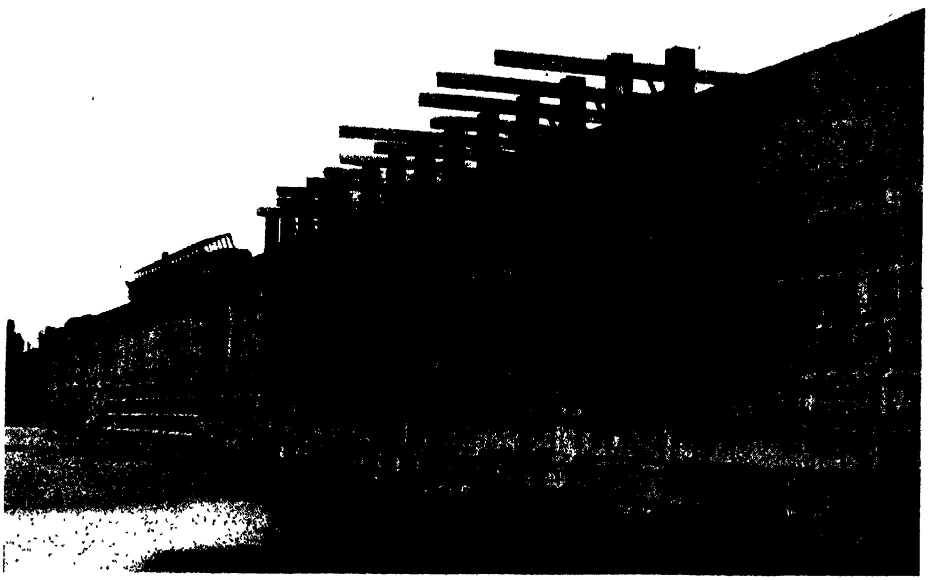


Fig. 2.

On the sea face and on the stepped back of the walls "Blawforms" were used as shuttering, with a special steel form for the curved nosing of the coping; this was supported by curved steel tee-irons and wedges as shown in *Fig.* 3. Normally the shuttering was supported from the timbering of the cofferdam, but on a short length where the cofferdam was damaged by very heavy seas it was necessary to erect rolled steel joists as soldiers (*Fig.* 3), strutted and bolted in pairs to carry the wall forms.

The mass concrete (quality A) used in

constructing the sea wall is in the proportions of 112 lb of rapid hardening Portland cement to 3½ cb ft of Rossett sand and 7 cb ft of Penmaenmawr granite passing through a 2½ in screen This is faced on exposed surfaces with a 12 in thickness of quality B concrete in the proportions of 112 lb of rapid hardening Portland cement to 2½ cb ft of sand and 5 parts of granite passing a 1½ in screen A shutter was not used to separate the two qualities of concrete

The method adopted to separate the two concretes as they were placed was to place a 12 in layer of B quality

The Blaw-Knox plant used on these contracts is illustrated in Fig 4 This comprised a steel bin batching plant with a 1 cb yd Rex mixer Concrete was delivered into 1 cb yd skips with bottom opening doors and detachable bodies and transported to the forms in trains hauled by petrol locos The output of the concreting plant when working to its full capacity is 60 cb yd hourly

In the sea wall there are plain butt expansion joints at intervals of 60 ft The joints in the coping are spaced 40 ft apart.

At intervals of 20 ft along the inside of the wall are partition walls 2 ft wide and

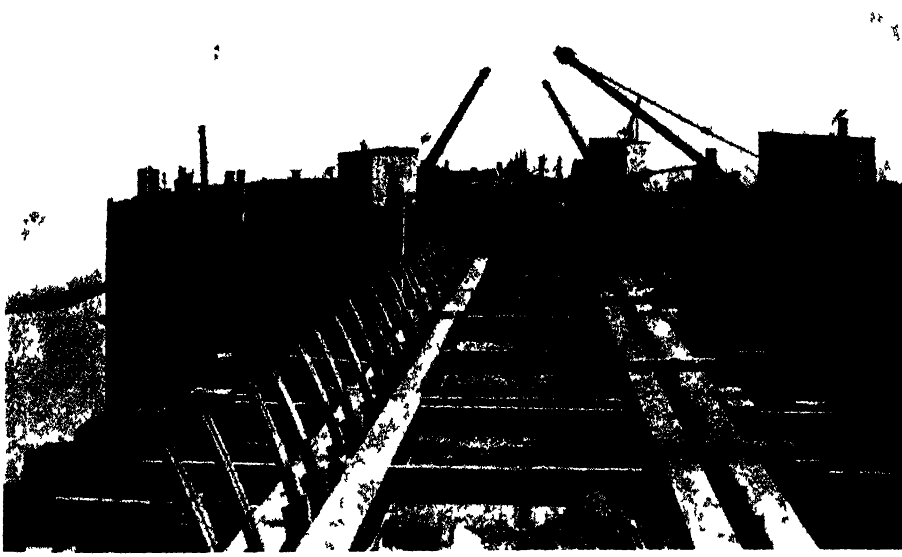


Fig 3

concrete adjoining the face of the wall and let this stand at its natural angle of repose following this up immediately with A concrete for the remaining width of the wall The whole of this concrete was then well punned and the process repeated in 12 in layers until a total lift of 4 ft was attained For the reinforced concrete work, including the precast piles the proportions used are 112 lb of rapid hardening cement to 2½ parts of Rossett sand and 5 parts of ¾-in Penmaenmawr granite In all concrete work the fine aggregate is a pit sand which is washed twice before being delivered on the site

above the promenade level these form spaces in which seats are being provided and anchored to 3 in square timber plugs 12 in long cast in the concrete Seven 12 in diameter outlets with flap and reflux valves are provided through the wall at a depth of 4 ft below the promenade level In addition the filling is drained by 9 in glazed earthenware open jointed pipes

The promenade includes a 50 ft and a 24 ft reinforced concrete carriageway separated by an intermediate footpath 10 ft wide In addition there are a 30 ft footpath between the sea wall and the 24-ft road and a 13 ft path alongside the

50 ft roadway The promenades are built on filling covered with 9 in of rock rubble The carriageways are composed of 8 in of concrete with B R C reinforcement and surfaced with a 2 in layer of asphalt and their total area is 28 000 sq yd Precast concrete kerbs supplied by the Liverpool Artificial Stone Co Ltd are being used and the footpaths are surfaced with concrete slabs supplied by Constone Ltd Of the latter there are 5 000 sq yd made with grey concrete and 22 000 sq yd tinted red The sub contractors for the kerb laying and

The site for the bath is 510 ft long by 410 ft wide On the south side of the bath is the championship area having a length of 165 ft and a width of 69 ft In this area the minimum depth of water is 5 ft 6 in at the east and west sides and a minimum depth of 15 ft is maintained in the middle of the area over a width of 33 ft for 10 metre diving To the north of the championship area the side walls are returned and the width of the bath is increased to 330 ft the depth of water in a 60 ft width on this length decreases uniformly from 5 ft at the southern edge

Fig. 4.

footpath construction are Messrs John McGeoch & Sons, of Wallasey

The Swimming Bath.

The open air swimming bath will be completed in May 1934, and will provide accommodation for 2 000 bathers and 7,000 spectators Among the modern features to be included are a sun bathing beach, under-water and flood lighting, and a deep water championship area The bath will be 330 ft long by 225 ft wide, and filtration plant will be installed capable of dealing with the whole contents of the bath in a period of eight hours

to 3 ft 6 in at the north Beyond this strip the water level decreases to zero on a width of 80 ft measured on the centre line of the bath The artificial beach has a maximum width of 25 ft The total water area is 52,400 sq ft and the contents of the bath are 1⅜ million gallons

The water supply will be obtained through two 21 in cast iron pipes from the marine lake which will act as a settling tank, and after passing through the filters the water will be pumped through concrete lined cast-iron mains with four 9-in. branches leading to the bath inlets A 12-in branch off these mains will feed an ornamental cascade The return from

the bath to the filters will be through a 24-in. concrete-lined cast-iron pipe.

Construction of the Bath.

The bath has been designed with mass concrete walls and a reinforced concrete two-course floor, the bottom slab of which is laid without expansion joints except where it meets the side wall. This slab rests on a 6-in. consolidated layer of rubble and is 6 in. thick for a width of 105 ft. extending from the northern edge of the bath towards the deep end. For the remainder of the slab the thickness is 12 in. The inner side of the wall is formed with a projecting footing, between which and the floor slab is a 1-in. thick vertical asphalt expansion joint. A strip of asphalt 2 ft. wide covers the junction of the lower slab and the projection of the wall footing. In both slabs the reinforcement consists of B.R.C. fabric, double reinforcement being used in the 9-in. upper slab and two layers of single reinforcement in the bottom slab where the latter is 12 in. thick. Before placing the top slab the upper surface of the bottom course is painted with bitumen.

In the upper slab two series of expansion joints have been formed dividing the slab into rectangular panels approximately 50 ft. by 30 ft. in area. These joints consist of a 2-ft. wide horizontal strip of asphalt 1 in. thick and a central 1-in. vertical thickness extending throughout the depth of the 9-in. slab. The floor is treated with a rendering of waterproofed cement and a finishing coat of white Portland cement, with a rough non-slip surface.

The bath wall is founded partly on clay and partly on sand, the portion surrounding the deep part of the bath resting on forty 14-in. square precast reinforced concrete piles driven at from 12-ft. to 13-ft. centres. In the piled length of the wall the heads of the piles are stripped for a length of 2 ft. in order to provide a bond between the reinforcement and the mass concrete of the wall. The minimum thickness of the wall is 1 ft. 6 in. and increases by steps 10 in. wide and 2 ft. deep; the wall footing is 12 in. thick. At the section on the centre line at the deep end the wall is 17 ft. high. A 12-in. by 3-in. cast stone non-slip coping is provided at the edge, and the faience scum channel is fitted with outlets at 30-ft. centres to a stoneware drain. The face of the wall is rendered with white waterproof cement.

The bath surround is a 6-in. reinforced concrete slab with an indented surface, laid to fall to a 12-in. by 1-in. surface channel discharging at gullies at 50-ft. centres.

The mass concrete in the bath wall is of the richer mixes previously mentioned, except in the deep-water diving area where quality A concrete faced with 12 in. of quality B was used. Expansion joints in the bath wall are six in number, and are formed of asphalt 1 in. thick, the front edge of each joint being caulked with lead wool. Concrete is placed in the floor by means of rectangular 1-cb. yd. bottom-opening skips hauled on jubilee track from the central mixing station and unloaded by a crane travelling on caterpillar track. The contract for the bath buildings has recently been let to Messrs. Wm. Tomkinson, of Liverpool.

British Standards Institution.

MR. MAURICE F. G. WILSON, M.INST.C.E., who has been chairman of the Main Committee and chairman of the General Council of the British Standards Institution for the past five years, has retired in accordance with the by-laws, and Mr. E. J. Elford, M.Inst.C.E., Borough Engineer and Surveyor of Wandsworth, has been elected to succeed him as chairman of the General Council for the next twelve months.

Combined Bending and Thrust or Tension.

By A. G. BOORMAN, D.I.C., M.Sc., A.M.Inst.C.E.

THE rapid selection of concrete sections to resist combined bending stresses and compression or tension presents some difficulties. It is believed that the following method of analysis will be found convenient, particularly when the bending moment does not depend upon the section chosen, it can also be used in other cases without difficulty, whether the load is tensile or compressive, and whether the whole section or only a small portion of the concrete is in compression.

Considering the case of such a section in which thrust and bending are combined (*Fig.* 1) and making the usual assumptions, it may readily be shown that

$$P = bdc\left[\frac{n'}{2} + p'(m-1)\frac{n'-g'}{n'} - pm^{*}\frac{1-n'}{n'}\right].$$

Assuming p, p', g' and m to be constant, and n' constant for the time being

$$P = \alpha bdc.$$

The equation remains the same in form when t is the limiting factor.

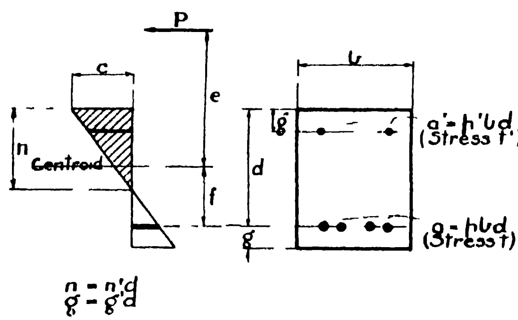


Fig. 1.

Taking moments about the steel in tension

$$M' = bd^2c\left[\frac{n'}{2}\left(1-\frac{n'}{3}\right) + p'\left(\frac{n'-g'}{n'}\right)(m-1)(1-g')\right]$$

$$= \beta bd^2c.$$

This is not to be equated to the external moment, for the latter must be taken about the centroid given by the equivalent area of the steel and the actual area of concrete in compression for any particular position of the neutral axis. This is, at first, a little difficult to visualise and has been overlooked in some analyses, but it may be illustrated as follows. Only the concrete in compression exerts a resistance, therefore that area only above the neutral axis is effective in fixing the centroid. The centroid is the position in which the resultant resisting load would lie if uniform strain could exist over the whole area and this portion of the

* This becomes $(m-1)$ when the neutral axis falls below the lower steel.

concrete only resist stress The true moment to which the external moment must be equated is

$$M \quad M' \quad Pf,$$

where f is the distance of the centroid from the tension steel and depends on the position of the neutral axis However, it will be found that

$$f = \frac{d\left[n' \quad \frac{n'^2}{2} + (m \quad 1)p'(1 \quad g')\right]}{mp + n' + (m - 1)p'} = \gamma d$$

Then
$$\begin{aligned} M \quad & \beta b d^2 c \quad \gamma b d c\, \gamma d \\ &= bd^2 c(\beta \quad \kappa\gamma') \\ &- \delta b d^2 c \text{ say} \end{aligned}$$

If therefore, there are two different concrete sections so loaded that n' and p p' and g' are the same then for a given limiting value of c the loads taken will be proportional to the dimensions of the section Thus

$$\frac{P_1}{P_2} \quad \frac{b_1 d_1}{b_2 d_2} \text{ and } \frac{M_1}{M_2} \quad \frac{b_1 d_1^2}{b_2 d_2^2}$$

Provided n' is the same for both its value is immaterial

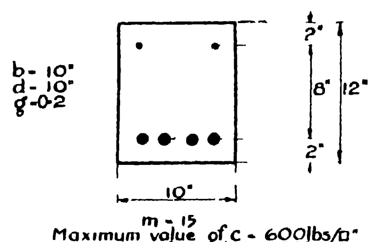


Fig 2.

It may be shown that this relation holds when compression exists over the whole section, or when a tension is combined with a bending moment

A rectangular concrete section, called a "reference section,' may be chosen, and the values of the external load and external moment calculated which may co-exist for all values of n' to give stresses that do not exceed the values of c or t selected Any other rectangular section of whatever dimensions, but having steel percentages and the value of g' equal to those of the reference section, may be called a "similar section," but will not generally be geometrically similar From the known properties of the reference section the properties of a similar section may be obtained by a single simple mathematical operation

As an example a reference section may be taken with the dimensions shown in *Fig.* 2 Here $b = 10$ in, $d - 10$ in, $g' - 0{\cdot}2$, $m = 15$, and the maximum values of c and t are 600 lb per square inch and 16,000 lb per square inch respectively These particular values need not be taken, but reference curves may be worked out for any figures which appeal to the designer The preparation of the curves will be found a little laborious, but once obtained will be equally useful to those given.

In *Figs.* 3, 4 and 5 are curves derived for the reference section, using various percentages of steel from 0·2 per cent. to 3·0 per cent. on the tension side and for the following conditions:

(*a*) Steel on the tension side only,

(*b*) Steel on the tension side equal to twice the steel area on the compression side, and

(*c*) Steel on the tension side equal to the steel on the compression side.

It may be found convenient to re-draw these in three different colours on

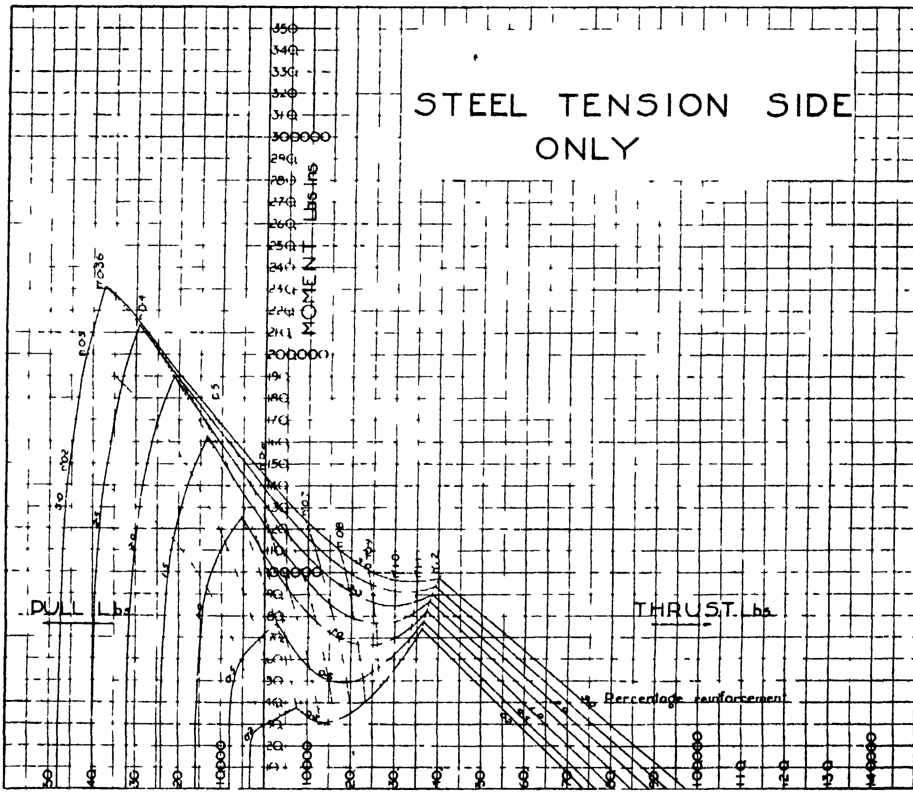


Fig. 3.

the same plan and to a larger scale, when it will not be difficult to interpolate both for steel percentages and for compression-to-tension steel ratios up to unity.

It will be observed that the method of plotting shows in a single diagram any combination of tension or thrust with bending moment, whether compression exists over the whole depth of the section or on part of it only. For sections having compression steel the curves have been stopped where the neutral axis has moved up to the steel line. In general, the position of the neutral axis should be noted when selecting a section.

The left-hand peak for each curve shows the point where the steel stress ceases to be the limiting factor (which is the case to the left of the peak), or where

the concrete stress ceases to be the deciding factor (which is the case to the right) For a small range of combinations only is it possible to choose a section in which both tension steel and concrete are simultaneously fully loaded The right-hand peak is the position where the whole area of concrete is in compression, as is the case also to the right of this point All curves are continuous through the moment axis This may be understood by considering the moment for low loads to be

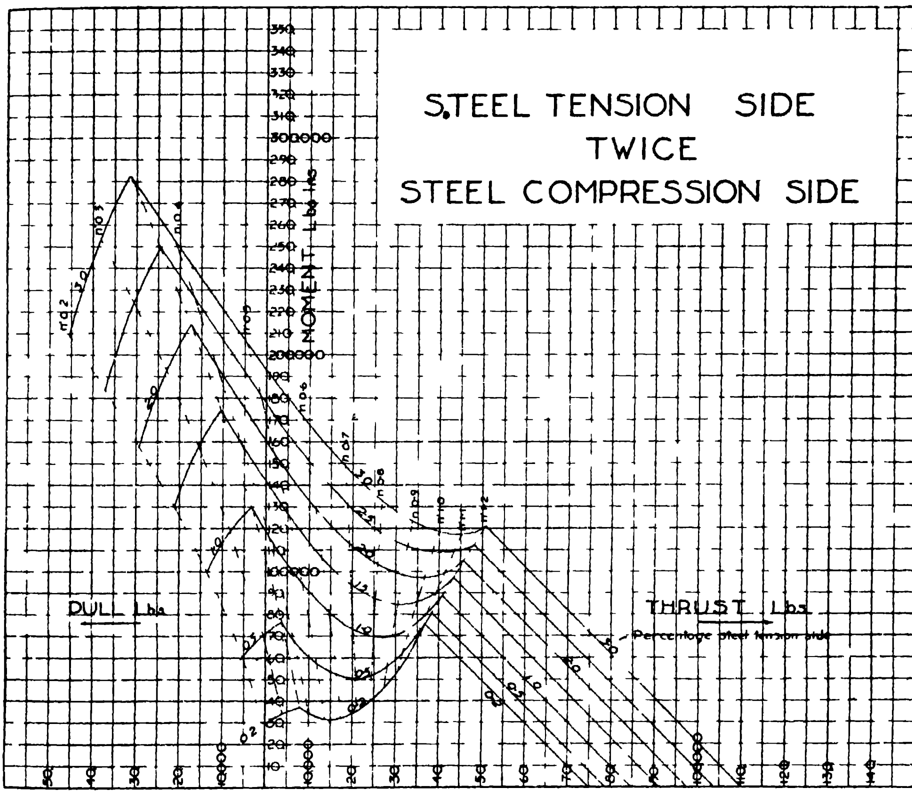


Fig 4

produced by a very small thrust acting at a very great distance The load gradually reduces, moving to an infinite distance, so that a finite moment is produced with zero load The thrust then changes sign to a very small tension and gradually returns, producing greater moments for gradually increasing loads

It must be noted that the whole of the section of reference has been used in calculating resisting forces and bending moments Where this is contrary to local regulations for certain members a set of curves may be calculated with this variation, but since the neglect of the concrete outside the steel only affects the side of the member which is in compression, and since here the steel will not be highly stressed, a close approximation may be obtained by using the reference curves to choose the section and then adding the thickness of fireproofing to the

dimensions. The steel in compression may then be moved outwards a small distance in order to make use of its full value.

The following example will assist to demonstrate the method.

To choose a section to carry 200 000 lb. thrust, and 2,000,000 lb. in. bending moment, assuming $c = 600$ lb. per square inch, $t = 16\,000$ lb. per square inch, and $m = 15$.

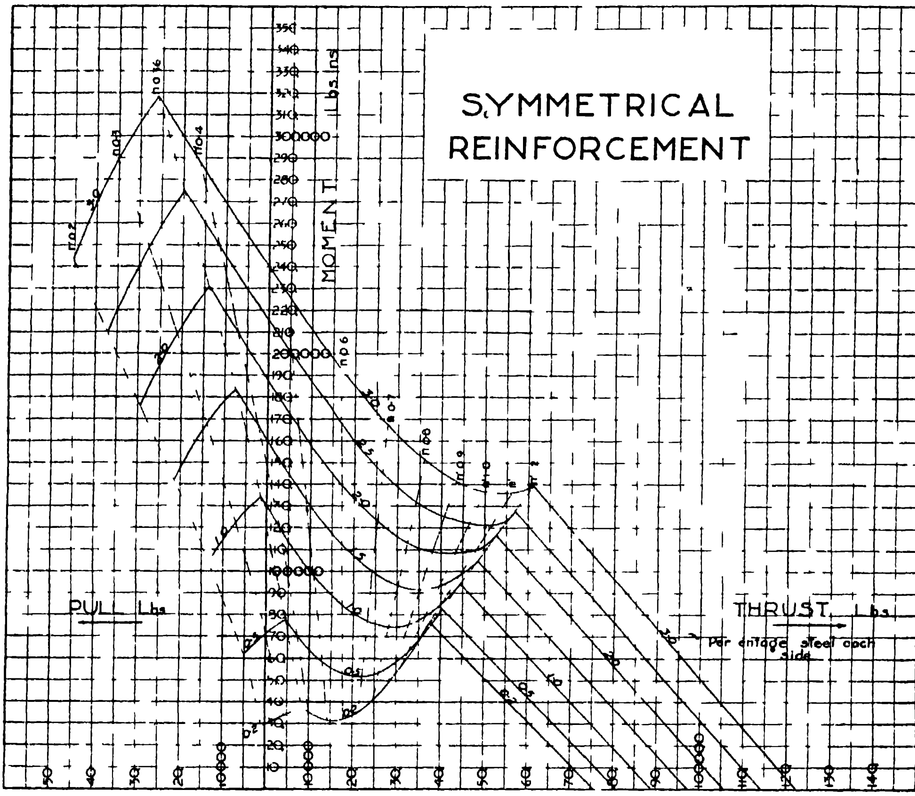


Fig. 5.

Try a section with $d = 30$ in. and $b = 20$ in.

Then the equivalent load (that producing equal stresses) on the reference section would be

$$P = \frac{200,000}{2 \times 3} = 33\,300 \text{ lb.}$$

$$M = \frac{2,000,000}{2 \times 3^2} = 111,000 \text{ lb. in.}$$

From inspection of the reference curves it is evident that the section would suit if reinforced with 2 per cent. on both sides, or with 2.5 per cent. on the tension side and 1.25 per cent. on the compression side, or with more than 3 per cent. on one side only (the latter being unsatisfactory).

Assuming a section with $d = 35$ in. and $b = 15$ in., the equivalent loads become

$$P = \frac{200{,}000}{1{\cdot}5 \times 3{\cdot}5} = 38{,}100 \text{ lb.}$$

$$M = \frac{2{,}000{,}000}{1{\cdot}5 \times 3{\cdot}5^2} = 109{,}000 \text{ lb. in.}$$

The following reinforcement will be suitable: (*a*) 2 per cent. on both sides, (*b*) 2·5 per cent. on the tension side and 1·25 per cent. on the compression side, or (*c*) more than 3 per cent. on one side only (the latter being unsatisfactory).

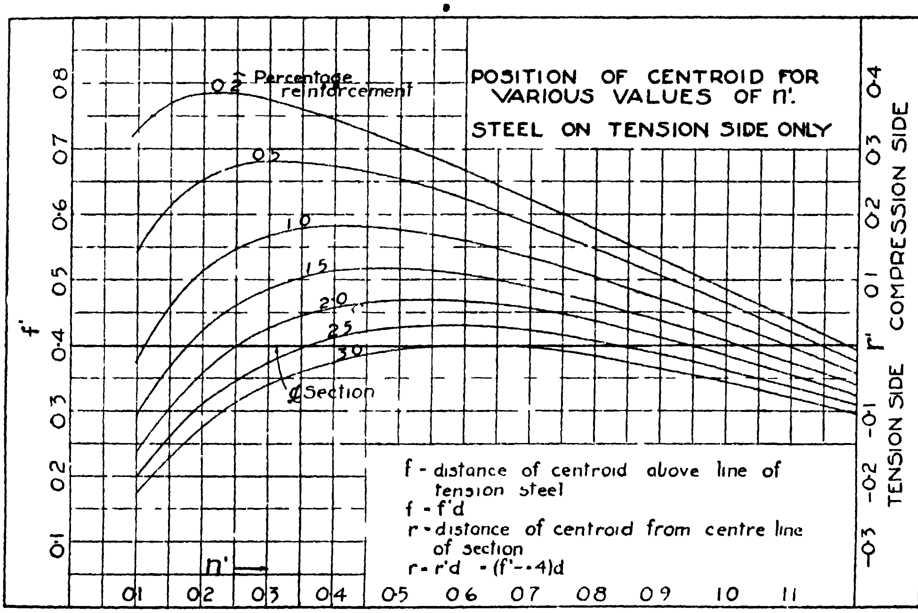


Fig. 6.

In this way different sections may be compared rapidly. Further, it is possible to use the curves to visualise the effect of variation in the external moment or load or in both.

Thus, for the case considered, choosing a 30-in. by 20-in. section reinforced with 2 per cent. of steel on both sides, if the thrust can fall to zero the section will be perfectly safe, even with a 70 per cent. greater bending moment. If the thrust is constant, any increase in bending moment will cause the safe stress in the concrete to be exceeded. If the bending moment remains constant, an increase in the load will only cause a small overstress in the concrete, even for a 60 per cent. increase in load.

For low percentages of reinforcement it must be noted that for sections chosen for low eccentricities, it is possible to render the section dangerous to the point of fracture by reduction of the thrust if unaccompanied by reduction in the bending moment. A similar effect is obtained if it is possible to reduce the load

without moment for sections subject to tension and bending, but this applies in the use of high reinforcement percentages.

Case where the eccentricity is known.

This occurs where the load to be carried is eccentric, either fixed in a certain position or necessarily at a certain distance outside the stanchion.

In the first case, if it is possible to move the stanchion so as to carry the load centrally, this should be done. If the eccentricity is very low (less than about 0·16d) compression will exist over the whole surface, and the centroid will be fixed for a fixed section, and will always be central for a symmetrically reinforced

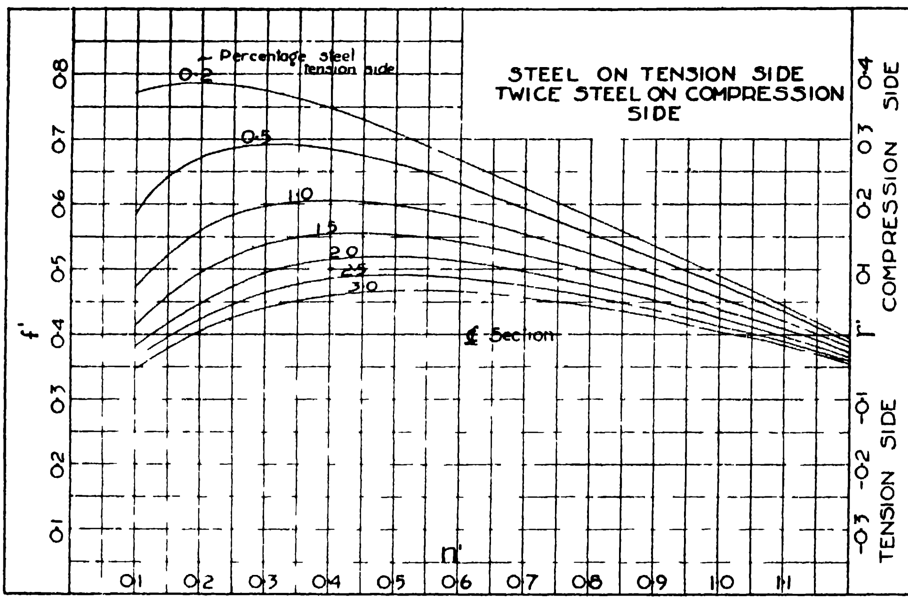


Fig. 7.

member. Thus choice of a trial section will immediately fix the eccentricity and therefore the bending moment, and no difficulty occurs.

In the second case the bending moment depends on the properties of the section; consequently for a first trial a position for the centroid must be assumed. In the case of thrust, if it is desired to use less than 2 per cent. of reinforcement, take the centroid at a distance from the centre line of the member towards the compression side of 0·1d when the thrust is within $\frac{3}{4}d$ of the tension steel, and at 0·2d when the thrust is farther than $\frac{3}{4}d$ from the tension steel.

If it is desired to have more than 2 per cent. of reinforcement, use 0·1d. In the case of tension (omitting instances where there is less than 1 per cent. of steel) take the centroid at a distance of 0·1d from the centre line of the member towards the compression side. Then the first trial section will fix the bending moment, and the curves may be used.

When suitable dimensions have been obtained, note the value of n' and,

from the centroid curves given in *Figs.* 6, 7, and 8, find the actual position of centroid. If this agrees with the assumption, the first trial is correct. If not, take the new position of the centroid and obtain the corresponding bending moment, and repeat.

Example.

It is desired to carry a load of 200,000 lb. from a bracket at a distance of 4 in. from the edge of the stanchion.

Try a section with $d = 30$ in. (overall measurement 36 in.) and $b = 24$ in. The eccentricity from the centroid is 18 in. + 4 in. – 3 in. = 19 in. (assuming that the centroid is at $0{\cdot}1d$ from the centre line on the compression side of the

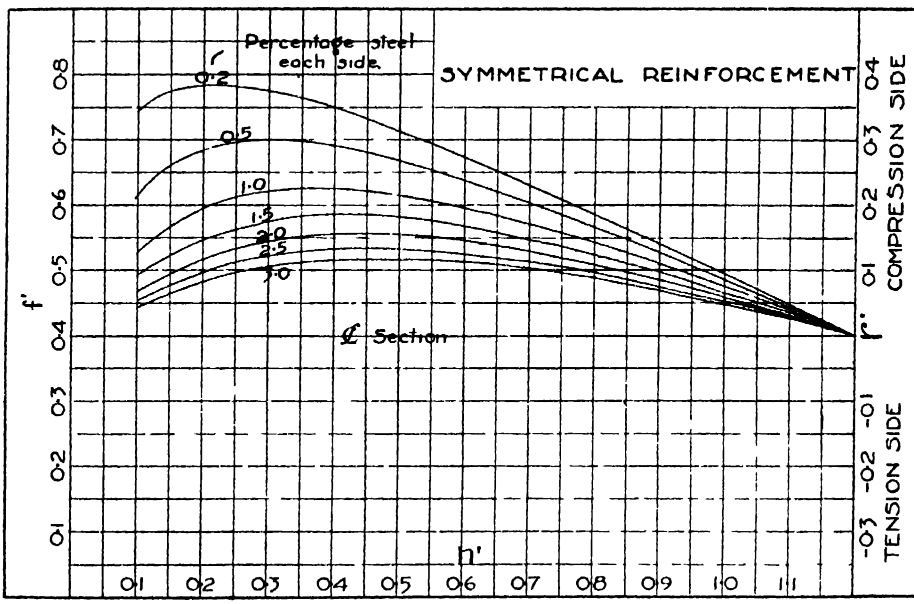


Fig. 8.

stanchion) and the bending moment is 19 × 200,000 = 3,800,000 lb. in. The equivalent loads on the reference section are

$$P = \frac{200,000}{3 \times 2{\cdot}4} = 27,800 \text{ lb.}$$

$$M = \frac{3,800,000}{3^2 \times 2{\cdot}4} = 176,000 \text{ lb. in.}$$

This requires more than 3 per cent. on both sides.

If we increase the width to 28 in.,

$$P = \frac{200,000}{3 \times 2{\cdot}8} = 23,800 \text{ lb.}$$

$$M = \frac{3,800,000}{3^2 \times 2{\cdot}8} = 151,000 \text{ lb. in.}$$

2·5 per cent. on both sides is just sufficient, and $n' = 0{\cdot}67$, $r' = 0{\cdot}118$, and $r = 0{\cdot}118 \times 30 = 3{\cdot}54$.

The actual eccentricity is $18 + 4 - 3{\cdot}54 = 18{\cdot}46$ in. and the actual bending moment is $18{\cdot}46 \times 200{,}000 = 3{,}690{,}000$ lb. in.

The bending moment being somewhat smaller than in the first trial, a 27-in. section width is tried.

The equivalent loads become

$$P = \frac{200{,}000}{3 \times 2{\cdot}7} = 24{,}700 \text{ lb.}$$

$$M = \frac{3{,}690{,}000}{3^2 \times 2{\cdot}7} = 152{,}000 \text{ lb. in.}$$

The percentage reinforcement for this width would require to be a little greater than 2·5.

It will be clear that if a square or flat section were desired, the first trial section would have been chosen accordingly; the process can be repeated to give any required shape.

Tests of Vibrated Concrete.

At the University of Stuttgart comparative tests of vibrated and hand-tamped concretes with various water-cement-ratios have recently been carried out by O Graff and K. Walz The vibration was produced by a " Pervibrator "—described in this journal in December 1930—a machine which floats in the freshly-placed concrete. The following notes on the tests have been compiled from an article in " Beton und Eisen "

Tests were made on columns approximately 12 in. square and 4 ft. 2 in. long and on 12-in. cubes. Two series of specimens were made, one containing about 400 lb. of Portland cement per cubic yard of finished concrete and the second containing 260 lb per cubic yard. The aggregates used were Rhine sand and gravel.

In the first series the proportions of the dry volumes of aggregates to 1 part of cement were 1·60 parts from zero to 1 mm., 0·80 parts from zero to 3 mm., 1·60 parts from 3 mm. to 7 mm., and 4 parts from 7 mm. to 30 mm. This series was sub-divided into two parts, A and B, by using different quantities of gauging water, the water-cement-ratios being 0·52 and 0·81 by weight respectively and the corresponding weights of cement in 1 cb. yd. of concrete 400 lb. and 401 lb. respectively.

The proportions of the volumes of dry aggregates in the second series were 2·60 parts from zero to 1 mm., 1·30 parts from zero to 3 mm, 2·60 parts from 3 mm. to 7 mm, and 6·50 parts from 7 mm. to 30 mm Two sub-divisions, C and D, were made with cement-water-ratios of 0·84 and 1·29 by weight and containing 260 and 255 lb. of cement per cubic yard.

After being stored for 7 days under damp cloth and 21 days in air the specimens were weighed and tested with the following results. The figures given are the average results for vibrated specimens and those enclosed in brackets are for tamped specimens.

Specimen	Density (lb. per cubic foot)	Compressive strength (lb. per square inch)
Column A	148 (140)	5400 (1960)
Cubes A	(140)	(2470)
Column B	153 (143)	2020 (2180)
Cubes B	(144)	(3180)
Columns C	148 (141)	2600 (1400)
Cubes C	(142)	(2060)
Columns D	146 (141)	738 (880)
Cubes D	(142)	(1350)

The Foundations of the Ford Motor Company's Power House at Dagenham.—III (*Concluded*).

By R. V. ALLIN, M.Inst.C.E.

Turbine Basement Walls.

The walls of the basement are of reinforced concrete designed as cantilevers 24 ft. high from basement to ground floor at + 20·00 O.D. (*Figs.* 30 and 30A). To allow a layer of asphalt to be put on at the back of the walls for their full height before the mass concrete was poured, a skin wall was necessary of sufficient thickness to embed the walings of the steel sheeting, which consisted of rolled steel joists. Owing to leakage of water through the sheet piling it was found impracticable to build this wall in concrete sufficiently watertight for satisfactory asphalting. A 9-in. brick wall backed with concrete was therefore substituted on which the asphalt was laid, drainage being effected as the work proceeded by omitting bricks where necessary. The reinforcement of the main wall was arranged in groups of six bars with intervals between the groups to facilitate the work of asphalting after the bars had been placed. The thickness of the wall between the face of the asphalt and the back of the 2-in. hollow space behind the 9-in. glazed brick facing is 6 ft. At the corners of the basement there occurs a change in the cantilever action of the walls due to the restraint of the end walls which is considerable, and small-scale experiments were made to determine the distribution of the stresses. *Fig.* 31 shows the arrangement of the diagonal reinforcement employed as the result of the investigation.

Pump House and Valve Chamber.

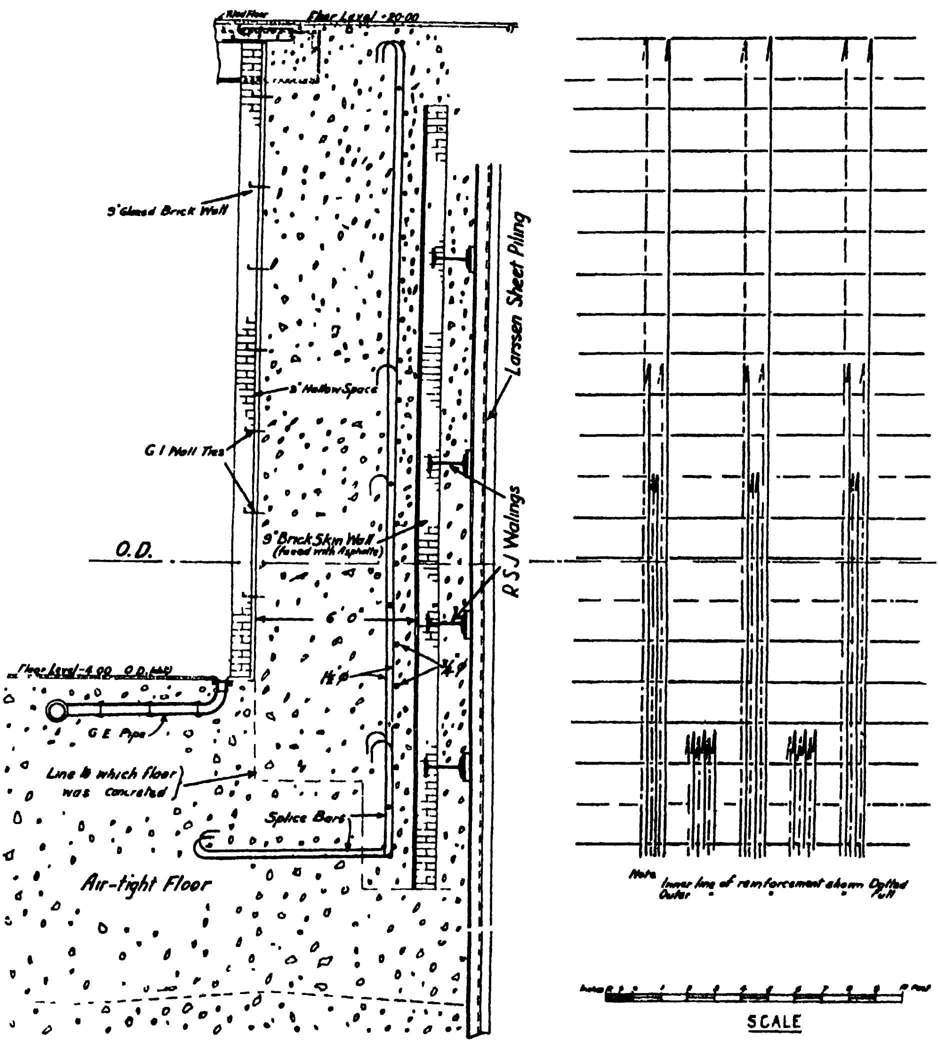
In this part of the basement the loads were such that the floor and walls could be safely carried by piles. The pump well was formed in the lower portion of a caisson 54 ft. long by 25 ft. wide, which was sunk in compressed air to a depth of — 35·25 O.D. and subsequently cut off above floor level (*Fig.* 32). After the caisson had been sunk the piles mentioned were cut down as the excavation around it proceeded. Those piles which had toe levels above that at which the caisson had been founded were re-hammered, and it was found that the material around them had been loosened by the sinking of the caisson to such an extent that some of them went down a further 14 ft. before showing the set to which they had originally been driven.

On the site of the pump well caisson No. 5 Larssen sheet piling, 80 ft. long, forming a part of the original scheme, had been driven. An endeavour was made to pull out this sheeting, but although a pull of 300 tons was applied by jacks, which stretched the pile 3 in. elastically, no permanent withdrawal resulted (*Fig.* 33). It was therefore necessary to burn off this sheeting in short lengths ahead of the cutting edge of the caisson as it was lowered. Both oxy-acetylene and the electric arc processes were used, but the former method proved more satisfactory.

In carrying out the excavation around the pump well caisson considerable trouble was experienced through the escape of air from the faces of the approaching tunnel into the ballast, the top of which was about 3 ft. below the lowest point in the excavation. This air chiefly came in through gaps between the

sheet piling and the outside of the intake shaft, and brought considerable quantities of water with it. Although it was possible to deal with the water, the air caused a considerable amount of trouble when pouring the concrete. This difficulty was met by leaving vent pipes in the concrete as a relief to the air pressure. These pipes were afterwards grouted up.

The chamber which contained the screens was formed in a steel caisson sunk under compressed air to a depth of - 36·25 O.D. There were two air-tight roofs in this caisson, the lower one over the working chamber and that required for constructing the openings into the pump well and intake shaft.



TYPICAL SECTION OF BASEMENT WALL. ELEVATION OF REINFORCEMENT.

Fig. 30.

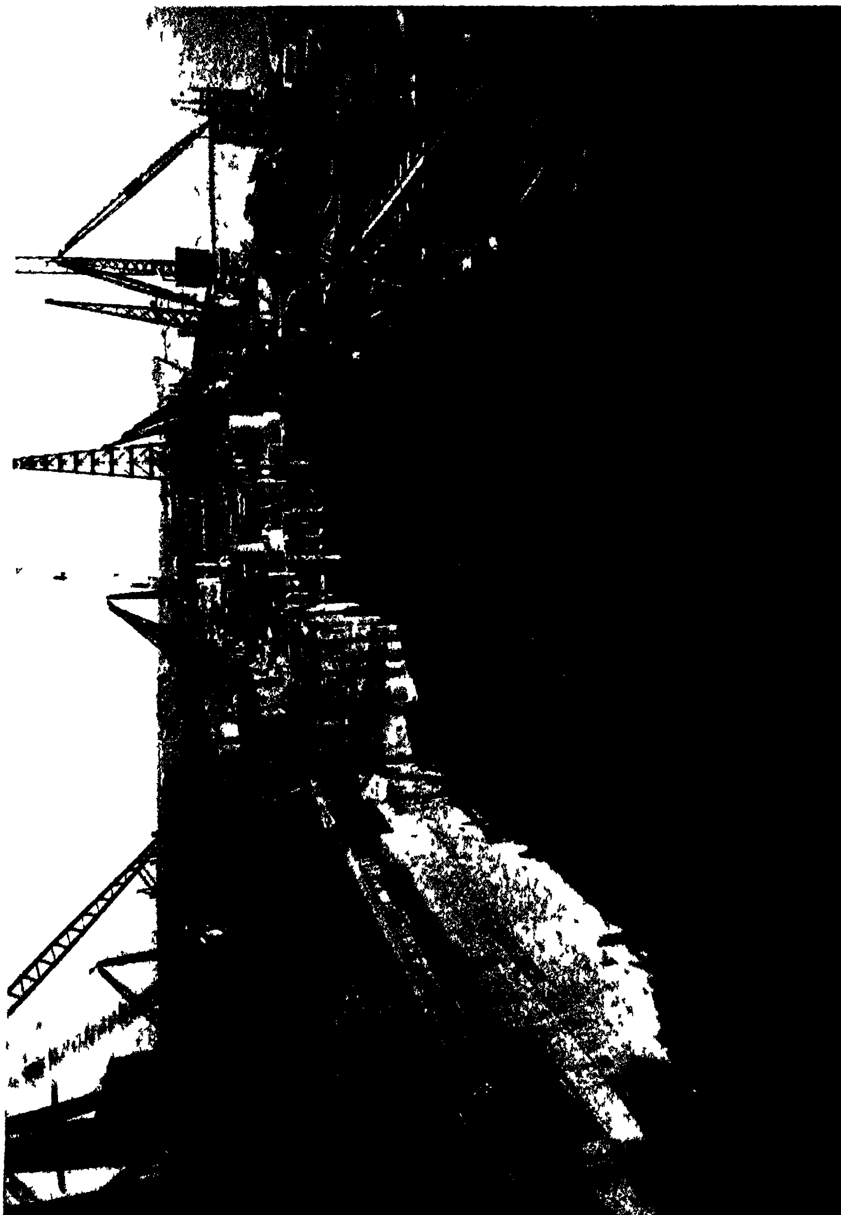


Fig. 30A.

The roofs were constructed of rolled steel joists and concrete, through which the two working shafts passed. When making the openings to the pump well caisson and intake shaft it was discovered that a water channel with a cross sectional area of about 2 sq. ft. had been formed around three sides of the caisson. This channel had been made by the air pressure from the tunnels passing through

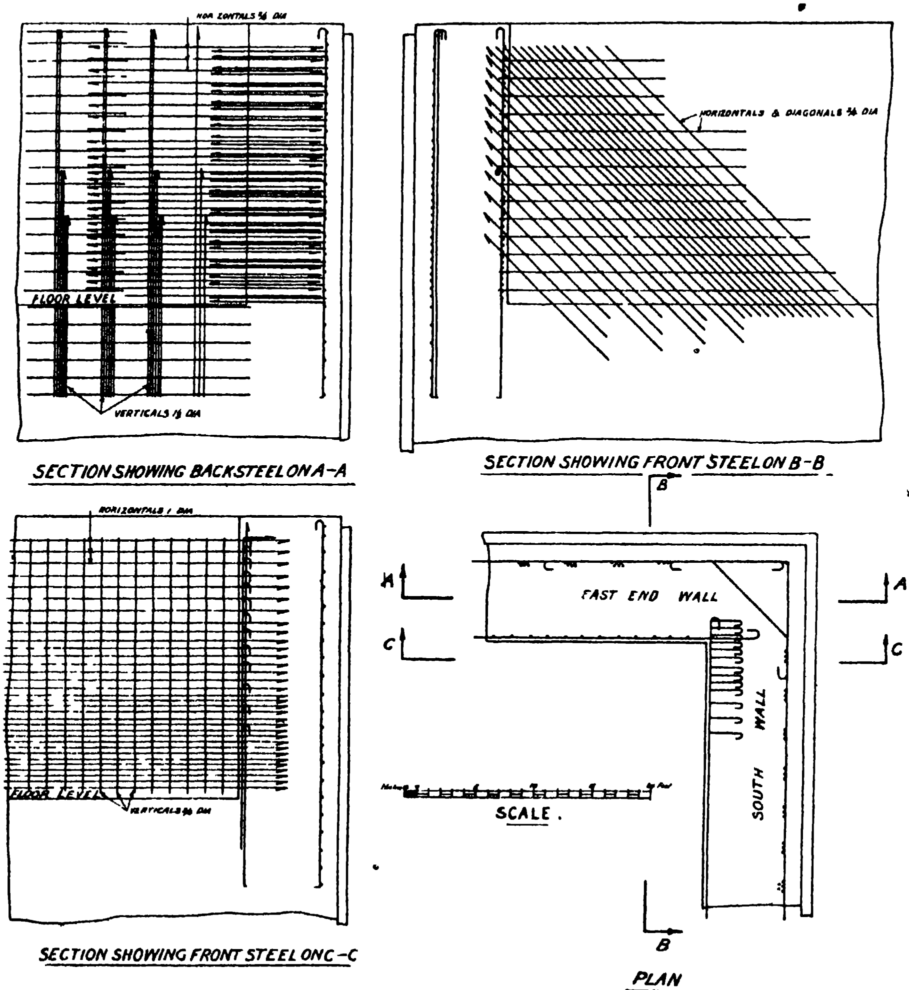


Fig. 31.

the soft peat immediately above the ballast (at about — 17·00 O.D.) into the excavation for the pump well basement mentioned. After sufficient air pressure had been put on to drive the water out of this channel it was filled with lime grout.

The concrete in the north-west wall of the north-west opening from the screen chamber into the intake shaft was found to be permitting a leak to the extent of about 350 gallons per hour at a level of about — 20·00 O.D. This

water, which came from the back of the wall, was under a head of about 20 ft. Messrs. John Mowlem & Co., Ltd., the contractors, successfully applied a chemical consolidation process to seal the voids in the concrete through which the water was passing. Briefly, this process consists in forcing into the concrete under pressure two chemical solutions successively which re-act on each other and form an amalgam which fills the interstices of the concrete and renders it watertight. In this instance six holes of about 2 in. diameter and about 2 ft. apart were drilled into the face of the concrete wherever water was entering

Fig. 32.

to depths and in directions dictated by experience. Solution No. 1 was pumped in under a pressure of about 300 lb. per square inch—with hand pumps—until it appeared in many places on the face of the wall; it was immediately followed by solution No. 2 at a higher pressure (600 lb. per square inch) to mix the chemicals. These operations were repeated until no signs of leakage appeared over a distance of 24 in. radius around the hole treated, and were continued until complete watertightness was attained.

The screen chamber was divided by concrete walls into four compartments for installation of two twin-flow screens and regulating penstocks.

The two land shafts consisted of ten steel strakes each 13 ft. 6 in. inside diameter. Each shaft had two compressed air working chambers, one for sinking the shaft and one immediately above it, into which the shield was received

at the end of its drive from the river shaft, through a 9-ft. diameter plugged opening. The sinking of the shafts was controlled in the earlier stages by hydraulic jacks supported on guide trestles and attached by links to the steel work (*Fig.* 35). Water kentledge controlled the subsequent sinking. The top of the shafts was at + 19·50 and they were founded at — 49·50 on dark sandy clay. The axes of the tunnels entered the shafts at – 36·00 O.D.

Concrete.

The concrete used on the work described was of the following qualities. Quality A: 112 lb. (1 bag) of rapid-hardening Portland cement to 6 cb. ft. of

Fig. 33.

Thames ballast (not exceeding 2½ in.); Quality B: 112 lb. of rapid-hardening Portland cement to 6 cb. ft. of Thames ballast (not exceeding 1½ in.); Quality C: 112 lb. of rapid-hardening Portland cement to 2½ cb. ft. of sand up to ¼ in., and 5 cb. ft. of coarse aggregate (¼ in. to ¾ in.); Quality E: 112 lb. of high-alumina cement to 2½ cb. ft. of sand up to ¼ in., and 5 cb. ft. of coarse aggregate (¼ in. to ¾ in.). Quality A was specified for mass concrete under compressed air floors generally; B for compressed air floors, the walls of the turbine basement, screen chamber and shafts, and pile caps; C for beams and slabs; and Quality E for precast piles.

It was specified that a ¼-cb. yd. sample from each delivery of Thames ballast should be screened and that the quantity of sand passing through a ¼-in. mesh should not be less than 40 per cent. or more than 65 per cent. of the ballast retained on the screen, and that if necessary sand or screened ballast should be added until the necessary proportions were obtained. It was further specified that in the event of the voids in the aggregate—after the addition of the sand

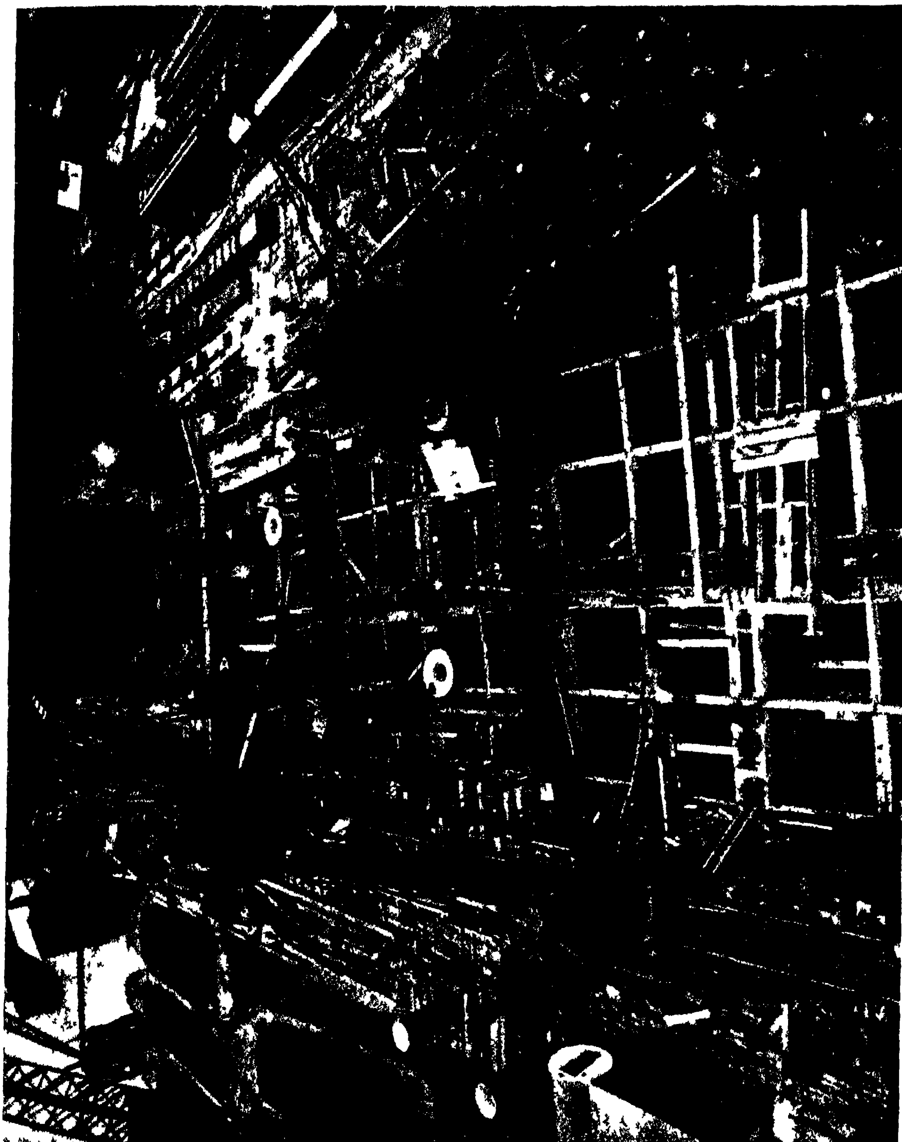


Fig. 34.

required exceeding 15 per cent, the proportion of cement should be increased until it was at least 25 per cent in excess of the voids

The following minimum crushing strengths for 6 in test cubes, seven days after casting, were specified Qualities A and B, 2,500 lb per square inch, Quality C, 3,000 lb per square inch, Quality E, 4,000 lb per square inch

Owing to the fact that there was no direct access by road to the site of the power house during the greater part of the construction, practically all the material and plant had to be water borne, and a timber wharf was built along the whole frontage of the site from which material was handled and placed by

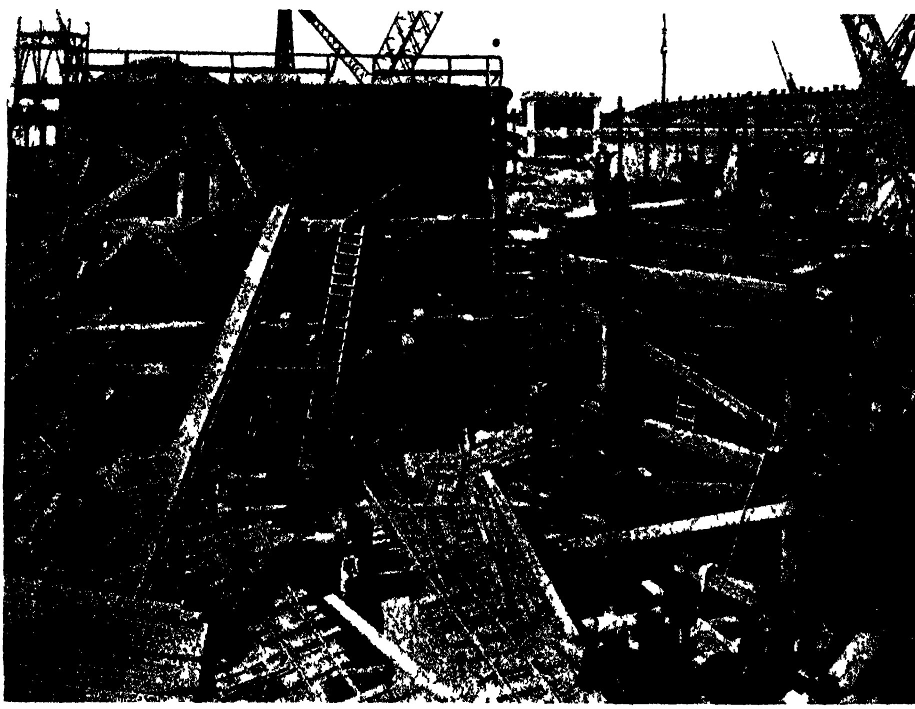


Fig 35

12-ton and 15-ton electric derricks (*Fig* 34) At the west end of this wharf was erected the main concrete mixing station (*Fig* 35) which consisted of three 1½-cb yd. batch mixers, electrically driven, supplied with aggregate from overhead hoppers from a 7-ton derrick that also unloaded it from the barges. The cement was brought to the mixer by a belt conveyor from a cement shed on the wharf After it was mixed the concrete was shot direct into bottom-dumping skips on wagons and hauled to the work by petrol locomotives on 24-in. Decauville track

The compressed-air installation for the work which has been described consisted of two rotary compressors with a capacity of 1,020 cb ft. of air per minute at 15 lb per square inch and two delivering 610 cb ft. per minute at 15 lb per square inch An emergency plant, comprising one of the larger com-

pressors coupled to a Diesel oil engine, was supplied for use in the event of a total failure of electric power. Ingersoll-Rand compressors supplied high-pressure air for tools. *Fig.* 26 shows the arrangement of the compressed air distribution.

In addition to the work described, this contract included two cast-iron tunnels 7 ft. 3 in. internal diameter subsequently lined with concrete, connecting two shafts sunk in the river with those already referred to, sunk at the power house. Circulating water for the electrical machinery and for use in the factory is drawn from and returned to the river in these tunnels, and dredging has been carried out so that the tops of the river intake and discharge shafts are kept at a level at which they will always be submerged. The shafts are provided with screens. A dividing wall of No. 3 Larssen sheet piling 200 ft. in length separates these shafts to prevent the discharged warm water from entering the intake tunnel, and both orifices are protected from shipping by a timber staging.

The works comprised in this contract were commenced in August 1930 and substantially completed in September 1932.

The author desires to express his thanks for the permission to write these articles given on behalf of the Ford Motor Co., Ltd., by Mr. J. H. Boyd, M.I.A.E., Chief Engineer of the Power and Construction Department, who was engineer for the whole of the works at Dagenham, and to Sir Cyril Kirkpatrick, PP.Inst.C.E., under whose direction as consulting engineer the works described in this paper were designed and constructed, and for whom during the latter half of the work the author acted as resident engineer. The main contractors for the works described were Messrs. John Mowlem & Co., Ltd.

(Concluded.)

Book Reviews.

"Mittig gedrückte Säulen. By A. Kleinlogel and K. Hajnal-Kónlji.
Berlin: W. Ernst & Sohn. Price: R.M. 4 60.

THIS is the second part of the authors' book dealing with examples of calculations in reinforced concrete design, and covers the various methods adopted in detailing axially-loaded columns with hooped or spiral binding. A section is also given treating the modifications required when the slenderness ratio of the column is greater than 15 for hooped columns or 13 for columns with spiral wrapping. The examples included are worked in detail.

"Behalter, Maste, Schornsteine, Rohrleitungen" ("Handbuch für Eisenbetonbau," Vol. IX), Parts 1 and 2. By Dr. F. Emperger.
Berlin: W. Ernst & Sohn. Price: R.M. 5.50 each.

THE first two parts of this volume, dealing with the design and construction of reservoir, poles, chimneys and pipes, have been written by Professor B. Loser, of Dresden, and carry the subject matter up to special classes of tanks required in certain industries.

The opening chapter deals with the materials used in making concrete tanks and the means adopted for securing impermeability. Considerable attention is given to special waterproofing and acid-resisting compounds. This is followed by a list of completed tanks and their dimensions and wall thicknesses. To English readers the drawings of details of reservoirs and tanks will prove useful, as indicating the principal features of German design. A large number of completed structures is shown, including reservoirs below ground, elevated water tanks, gasholder tanks, and swimming baths. Many of these are of unusual dimensions and recent construction.

Reinforced Concrete Residential Flats.

At Cornwall Gardens, Kensington, S.W., a block of flats has recently been completed to the design of the Estate Department of the Prudential Assurance Co. This building is noteworthy as being the first of its kind erected in London in which the whole of the structural work is executed in reinforced concrete. The elevations (*Fig.* 1) are in white Portland cement concrete. At fourth floor level there is a balcony with reconstructed stone corbels and tympanum.

There are twenty-four residential flats divided into three types according to the accommodation provided. In the basement are the coal and coke stores, luggage store, electricity sub-station room and meter room, store rooms, porters' room, and a flat for the resident porter. Access to the basement is by stairs and lifts.

At ground-floor level is the main entrance hall which extends as a central hall from front to back. Four flats are provided on this floor. Each flat is situated at a corner of the rectangular block. The planning of the first, second and third floors is similar, with four flats on each floor. At fourth-floor level a wide verandah has been built on the north end of the block, and a narrower verandah on the south and east sides.

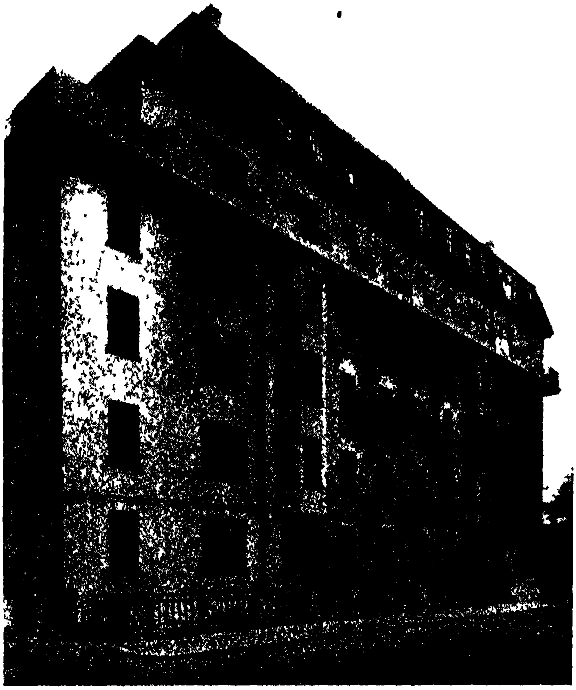


Fig. 1.

Construction.

The foundations were taken down to the London clay, and consist of isolated reinforced concrete footings. These are square in plan, their dimensions being proportional to the column loads. The reinforcement consists of a mat of ⅝-in. bars at 6-in. or 7-in. centres. The columns are square in cross section, the largest having a 27-in. side and being reinforced with twelve 1¼-in. vertical bars and links. This column stands on a pyramid-shape footing 11 ft. by 10 ft. 3 in. in plan having a 9-in. reinforced slab at the bottom. In the columns and footings the proportions of the concrete are 1 : 2 : 4.

Below the basement floor are special

floor beams continuous between two columns and with cantilever extensions to carry the party wall without bringing the exterior row of column footings close to the line separating the two buildings In a similar manner cantilever beams were inserted in the basement to carry a portion of the building over the Metropolitan Railway tunnel at the junction of Stanford Road and Cornwall Gardens was adopted the design was altered to take advantage of the reduced loading (40 lb per square foot) permitted under the code The floors were constructed in concrete mixed in the proportions of 1 1½ 3 The construction consists of main tee beams of reinforced concrete between which the hollow tile floors span Typical main beams are 11 in deep overall and 7 in wide in the web with

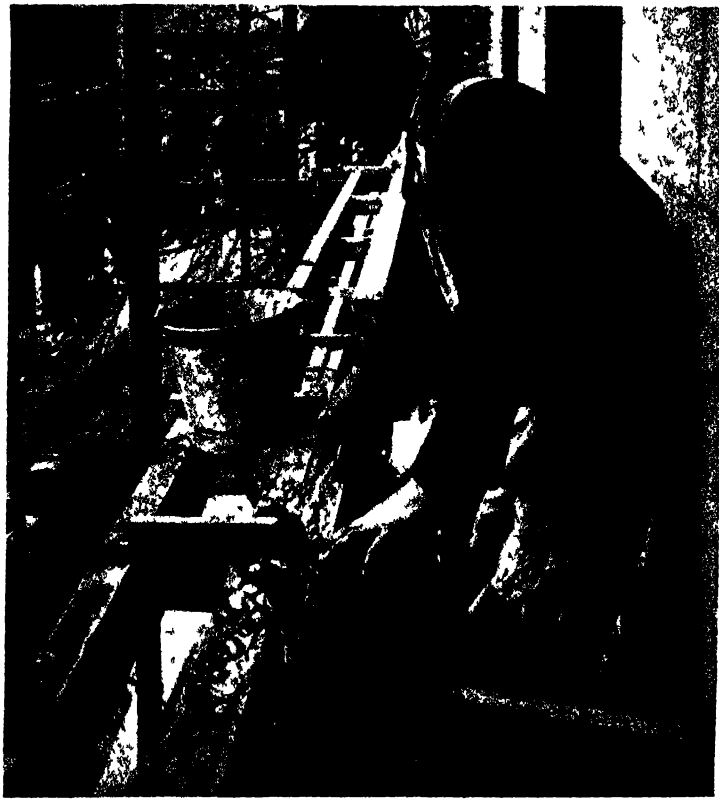


Fig. 2.

When the original drawings were made it was not expected that this tunnel would be met with in the excavation consequently a re-arrangement in the design of the basement became necessary and the difficulties of construction were considerably increased during the period from June to November of last year

Originally the floors were designed to carry a superimposed load of 70 lb per square foot as required by the L C C Regulations, but after the "Steel Code" flanges 17 in wide, the cross sections of the beams varying with the spans Between these beams the hollow-tiled floor consists of 10 in by 4½ in tiles at 13-in centres, giving a 3 in space to form the web of the secondary reinforced concrete beams The flanges of the latter are 2 in deep, the tops of the tiles being set at this distance below the floor level, and are reinforced by ¼-in bars at 12-in centres All suspended floors were screeded for wood-block surfacing

The stairs are constructed of reinforced concrete cast *in situ*, and finished in granolithic on which carpeting has been laid.

Walling.

There are two classes of interior walling, (1) a 2½-in hollow tile wall plastered on both sides for use as internal partitions, and (2) for the partition walls between pairs of flats a wall consisting of two skins of 2½-in hollow tiles with a 2-in air space between, and 2½-in. moler solid blocks supplied by Messrs. Sankey for the staircase partitions. Partitions are constructed directly on the floor as required.

The exterior walling consists of a 4½-in. reinforced concrete slab, a 2-in air space and a 2½-in. inside tile with plastering; the materials on the inner side of the 4½ in. reinforced concrete slab are required to conform to the L.C.C. requirements for fire-resisting construction, the slab itself being capable of resisting any possible combination of structural and wind loading. In the reinforced concrete slabs the reinforcement consists of a mesh of ⅜-in. horizontal bars at 12-in. centres and ¼-in. vertical bars at the same centres.

Fig. 2 illustrates the construction of a portion of the walling on the balcony at the fourth floor level. A sheet metal moving shutter was used to separate the white concrete facing from the grey concrete backing. The front and back forms for the wall consisted of ⅝-in. waterproof plywood built into panels with stiffened edges. These proved very successful and some were used as many as fifty times before they were scrapped. The thickness of the ordinary concrete is 4 in. and the white Portland cement concrete facing is 1 in. thick. The steel plates used were $\frac{1}{16}$-in. thick, 6 ft. to 7 ft. long, and 18 in high. Both the facing and backing concrete were placed in 18-in. lifts before the men raised the steel plates to allow them to bond together.

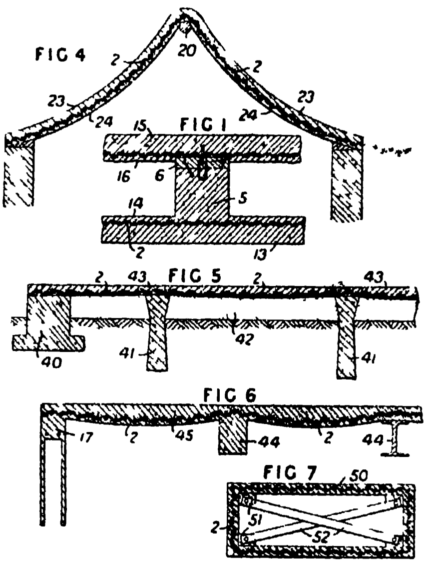
The contractors were Messrs. Hall, Beddall & Co, Ltd., and Messrs. F. Bradford & Co., Ltd, designed and constructed the reinforced concrete work. The steel reinforcement was supplied by the United Steel Co., Ltd.

Recent Patents Relating to Concrete.

Structures of Reinforced Concrete.

376,259.—J H de W Waller, 115, Grafton Street, Dublin March 28, 1931

A method of building with reinforced concrete, cement, plaster, or like material comprises stretching woven flexible material composed solely of vegetable fibres, substantially in a straight line between supporting means to give it an initial tension and then applying one or more layers of concrete cement plaster, or the like to the flexible material which forms



Structures of Reinforced Concrete

the reinforcing means and is kept under permanent tensile stress in the finished structure In the double wall shown in horizontal section in *Fig* 1 hessian (2) for the outer wall is secured at one end to a rail (not shown) supported by concrete pillars (5) and after being stretched is fastened to one or more other supports according to the length of the wall The hessian is preferably wetted to shrink it and is then covered with a thin paste of concrete or cement, and when dry is covered with layers (13, 14) to form the outside wall The hessian for the inner wall is fastened to timber strips (6) secured to the pillars (5) and when stretched in place is covered with plaster (15) which, is pressed through the meshes of the hessian to form a coating (16) The pillars (5) may be of timber

In the roof shown in *Fig* 4, hessian (2) is maintained in a stretched state over a ridge pole (20) Wire netting or other metal reinforcement may be placed under the hessian to prevent it sagging A layer of cement or concrete plaster (23) is placed over the outside of the hessian, and when this layer has set a layer (24) is placed on the inside, covering both the hessian and the wire netting

The application of the invention to floors is indicated in *Figs* 5 and 6 In the construction shown in *Fig* 5, the floor is adapted to be supported by footings (40) and concrete supports (41) A layer of sand (42) is spread over the ground so as to be level with the supports and the hessian (2) is laid on top and when stretched is secured to the footings (40) Concrete (43) is then applied to the hessian to form the floor surface

An upper floor may be made as shown in *Fig* 6 the hessian (2), which is stretched over the floor joists (44) and fixed to the top of the wall plate (17), being covered with concrete (45)

Floors or walls may be built from units, such as shown in *Fig* 7, comprising hollow tubular members (50) reinforced with hessian (2) The members are built around a framework of parallel members (51), preferably of timber, which are held in place by readily removable bracing (52) To form a floor, the members (50) are laid side by side the joints are grouted, and the whole covered with concrete, the undersides being, if desired, plastered to form a ceiling A space may be left between any two tubular members, and a reinforcing bar placed in this space which is filled with concrete to form a joist To form a wall, the tubular members are laid horizontally or vertically and bonded together by cement mortar, the outside and inside being covered with a thin layer of concrete, cement, plaster, or like material The invention may be applied to the construction of beams and stairs

Obituary.—We regret to announce the death of Mr Cecil Sainsbury who was with Messrs Stothert & Pitt, Ltd, for 43 years, and for many years joint managing director of the Company.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	7	9
Clean Shingle, ¾ in. mesh	,,	6	9
,, ,, ½ in. mesh	,,	9	0
Thames ballast	,,	6	9
Broken brick (¾ in.)	,,	10	6

Best British Portland Cement (delivered London area) per ton 46*s*., including non-returnable paper bags; 44*s*. 9*d*., including charge for hire of jute sacks.

Rapid-Hardening Portland Cement . . delivered London 7*s*. 6*d*. per ton extra.

"Colorcrete" red and buff rapid-hardening Portland Cement, delivered London 20*s*. per ton extra

Coloured cement and aggregate mixtures . delivered London area £5 to £5 10*s*. per ton.

"Snowcrete" White Portland Cement, £8 5*s*. per ton delivered London, including non-returnable paper bags.

"Super-Cement" Waterproof Cement . . . (including paper bags) per ton 76*s*.

Boarding for Shuttering—

		Sawn.	*Wrot.*
1 in	per square	22*s*. 6*d*.	26*s*. 6*d*.
1¼ in.	,,	28*s*. 6*d*.	32*s*. 6*d*.
1½ in.	,,	34*s*. 0*d*.	40*s*. 0*d*.

Sawn Timber for Struts and Supports—

3 in. by 4 in.	from £19 per standard
3 in by 6 in. and 3 in. by 7 in.	,, £18 ,, ,,

Mild Steel Rods for Reinforcement—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	8	6
9/16 in. to ½ in. Rounds	,,	8	9
⅜ in. Rounds	,,	9	0
¼ in. Rounds	,,	10	0

Breeze Slabs per yd. super: 2 in., 1/6; 2½ in., 1/8; 3 in., 2/–; 4 in., 2/4.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(*Based on Contracts up to £2,000.*)

Portland Cement Concrete well mixed in proportion 1 : 2 : 4—

				s.	d.
Do.	do.	in foundations	per foot cube	1	3
Do.	do	in columns	,, ,,	1	5
Do.	do.	in beams	,, ,,	1	7
Do.	do.	in floor slabs 4 in. thick	per yard super	3	8
Do.	do.	in floor slabs 5 in. thick	,, ,,	4	7
Do.	do.	in floor slabs 6 in. thick	,, ,,	5	6
Do.	do.	in floor slabs 7 in. thick	,, ,,	6	5
Do.	do	in walls 6 in. thick	,, ,,	5	6

(*Add for hoisting 3s. 6d. per yard cube above ground-floor level. Add for rapid hardening Portland Cement 2s. per yard cube.*)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	16	0
,, 7/16 in. to ½ in.	,,	15	0
,, 9/16 in. to 2⅞ in.	,,	14	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d*.; ⅜-in. rods, 1*d*.; ½-in. rods, 1¼*d*.; ⅝-in. rods, 1½*d*; ¾-in. rods, 1¾*d*.; ⅞-in. rods, 2*d*.; 1-in. rods, 2¼*d*.; 1⅛-in. rods, 3*d*.; 1¼-in. rods, 3½*d*.; 1⅜-in. rods, 4½*d*.; 1½-in. rods, 6*d*

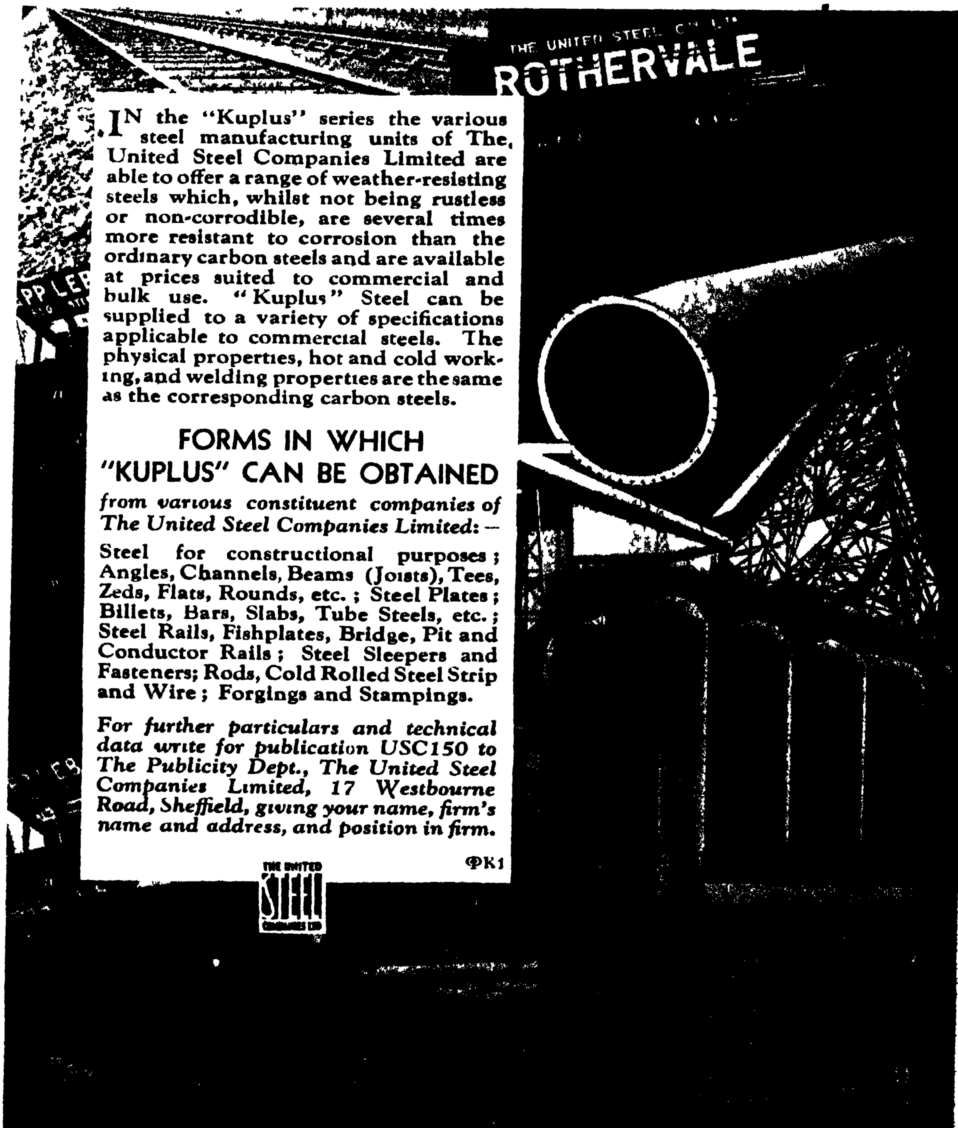
EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d*.; ⅜ in., 2*d*.; ½ in., 2½*d*.; ⅝ in., 3*d*.; ¾ in., 3½*d*.; ⅞ in., 4*d*; 1 in., 4½*d*.; 1⅛ in., 6*d*.; 1¼ in., 7*d*.

Shuttering—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	50	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	46	0
Do. do. in small quantities	per ft. super	0	8
Shuttering and Supports to Stanchions, average 18 in. by 18 in.	per ft. super	0	7½
Do. do. as last, in narrow widths	,, ,,	0	9½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	,, ,,	0	9½
Do. do. as last, in narrow widths	,, ,,	0	10
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/7 per hour; Carpenters working on old shuttering, 1/8; Labourers on building works, 1/2½; Men on mixers and hoists, 1/3½; Bar-benders, 1/3½.

'KUPLUS' WEATHER-RESISTING STEEL



IN the "Kuplus" series the various steel manufacturing units of The United Steel Companies Limited are able to offer a range of weather-resisting steels which, whilst not being rustless or non-corrodible, are several times more resistant to corrosion than the ordinary carbon steels and are available at prices suited to commercial and bulk use. "Kuplus" Steel can be supplied to a variety of specifications applicable to commercial steels. The physical properties, hot and cold working, and welding properties are the same as the corresponding carbon steels.

FORMS IN WHICH "KUPLUS" CAN BE OBTAINED

from various constituent companies of The United Steel Companies Limited:—

Steel for constructional purposes; Angles, Channels, Beams (Joists), Tees, Zeds, Flats, Rounds, etc.; Steel Plates; Billets, Bars, Slabs, Tube Steels, etc.; Steel Rails, Fishplates, Bridge, Pit and Conductor Rails; Steel Sleepers and Fasteners; Rods, Cold Rolled Steel Strip and Wire; Forgings and Stampings.

For further particulars and technical data write for publication USC150 to The Publicity Dept., The United Steel Companies Limited, 17 Westbourne Road, Sheffield, giving your name, firm's name and address, and position in firm.

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Prospective New Concrete Work.

ABERDEEN.—*Quay Wall.*—The harbour authorities have approved a scheme for reconstructing the quay wall at an estimated cost of £8,000.

ALLHALLOWS.—*Concrete Pool.*—The Hoo R.D.C. has approved a proposal to construct a concrete swimming pool at Allhallows.

ARBROATH.—*Bathing Pool.*—The T.C. has decided to construct a sea-water bathing pool at Seaforth House at an estimated cost of £8,000.

BEDWORTH.—*Swimming Baths.*—The U.D.C. proposes to construct open-air swimming baths at the Heath Road recreation ground.

BEESTON.—*Flood Prevention.*—The Beeston U.C. proposes to construct flood prevention works at Pasture Dyke at an estimated cost of £17,000.

BICKERTON.—*Water Supply.*—The Nantwich R.D.C. proposes to construct water supply works for Bickerton at an estimated cost of £12,200.

BIRMINGHAM.—*Flood Prevention.*—The T.C. has applied for a loan of £9,500 for flood prevention works at Park Hill, Harborne.

BODMIN.—*Swimming Pool.*—The T.C. is considering the construction of a swimming pool.

BOOTLE.—*Sewerage.*—The T.C. has approved a drainage scheme for the Rimrose Brook district at an estimated cost of £282,000.

BOSTON.—*Bridge.*—The T.C. is considering the widening of Bargate Bridge, on the Boston-Skegness road.

BRENTWOOD.—*Sewage Disposal.*—The U.D.C. has received sanction to borrow £26,447 for the extension of the Nag's Head Lane sewage disposal works.

BURNTISLAND.—*Swimming Bath.*—The T.C. is considering a proposal to construct a swimming bath at an estimated cost of £8,000.

CARDIFF.—*Weir.*—The T.C. proposes to construct a weir in the river Taff.

CONGLETON.—*Swimming Pond.*—The T.C. is considering proposals for constructing an open-air swimming pond.

CORBY.—*Water Tower.*—The Kettering R.D.C. is to construct a water tower near Rockingham Road with a capacity of 123,000 gallons.

CORNFORTH.—*Flood Prevention.*—The P.C. proposes to carry out flood prevention works at West Cornforth.

CROMER.—*Groyne.*—The U.D.C. is to construct a groyne at the east boundary of the foreshore at an estimated cost of £2,250.

DOVER.—*Road.*—The T.C. is considering a proposal to widen Archcliffe Road at an estimated cost of £8,000.

DUNDEE.—*Swimming Pool.*—The B.C. is proposing to construct a swimming pool on the beach at an estimated cost of £35,000.

DUN LAOGHAIRE.—*Swimming Pond.*—The T.C. is considering the construction of an open-air swimming pond.

DUNOON.—*Bathing Pool.*—The T.C. is to proceed with the construction of a bathing pool at Moir Street.

GREAT EDSTONE.—*Reservoir.*—The Kirbymoorside R.D.C. proposes to construct a reservoir at Great Edstone.

GRINDLETON.—*Bridge.*—The Bowland R.D.C. has approved the construction of a bridge and approaches at Grindleton, at an estimated cost of £5,000.

HILLSEA — *Sea Wall, etc* — The Portsmouth T C has applied for sanction to borrow £10 650 for a sea wall and £22 670 for the construction of a swimming bath at Hillsea

LAMBOURN — *Water Supply* The Hungerford R D C is proposing to undertake a water supply scheme for the Lambourn district at an estimated cost of £21 763

LLANELLY — *Swimming Pool* The T C is considering the construction of a swimming pool at the Lake Lands Parc Howard

LONDON (WOOD GREEN) — *Swimming Pool* — The U D C has applied for sanction to a loan of £26 104 for the construction of an open air swimming pool at Durnsford Road

LONG EATON *Sewage Disposal* — The U D C has applied for a loan of £42 750 for sewerage and sewage disposal works at Stapleford

LUTON *Sewage Disposal* The T C has applied for sanction to borrow £4 000 for sewage disposal works

MANBY — *Aerodrome* — The Air Ministry is to construct an aerodrome at Manby near Grimsby

MARCHLYN *Water Supply* The Gwyrfai R D C has approved a water supply scheme for Llanfairisgaer Llanrug and Llanddeiniolen at an estimated cost of £15 000

MARGATE *Reservoir* — The T C proposes to erect a reservoir with a capacity of 5 000 000 gallons

MARGATE *Sea Wall* - The T C proposes to complete the sea wall and promenade at Foreness Bay

MERTHYR — *Bridge* — The T C is to reconstruct Brandy Bridge

MINEHEAD — *Swimming Bath* — The U D C is to proceed with the construction of a swimming bath

MOUNTAIN ASH *Swimming Bath* The U D C proposes to construct a swimming bath

NANTWICH — *Open air Bath* — The U D C has approved the proposed construction of an open air swimming bath on the Salt Ley Meadow, at an approximate cost of £4 000

NANTWICH — *Water Supply* — The R D C is considering a scheme for the construction of a reservoir pumping station and ancillary works in connection with a water supply for Bickerton at an estimated cost of £12 200

NEWBIGGIN — *Breakwater* — The U D C is considering the construction of a breakwater

RAMSGATE.—*Water Supply.*—The T.C. has applied for sanction to borrow £12,500 for the extension of the water adits.

RICHMOND (SURREY).—*Swimming Bath.*—The T.C. is considering a scheme for the construction of an open-air swimming bath.

SALFORD.—*Bridge.*—The T.C. has decided to reconstruct Cromwell Bridge, connecting Pendleton and Broughton, at an estimated cost of £30,000.

SHOREHAM-BY-SEA.—*Bathing Pool.*—The U.D.C. proposes to construct an open-air swimming pool at the Adur Memorial Recreation Ground.

SHREWSBURY.—*New Road, etc.*—It is stated that the T.C. has decided to construct a new loop road from Welsh Bridge to Shoplatch, and to extend the car park at Barker Street to allow for an omnibus station.

SOUTHAMPTON.—*Sewage Disposal.*—The T.C. has applied for sanction to a loan of £236,000 for sewerage and sewage disposal works.

STONEHAVEN.—*Swimming Pool.*—The T.C. is to construct a swimming pool at an estimated cost of £5,500.

SUNDERLAND.—*Roads.*—The T.C. has received sanction to a loan of £23,035 for road reconstruction work.

TAMWORTH.—*Sewage Disposal.*—The T.C. and the R.D.C. have applied for loans of £4,790 and £3,702 respectively for the construction of sewerage and sewage disposal works.

THIRSK.—*Water Supply.*—The R.D.C. is to apply for a loan of £4,720 for water supply works.

TOTNES.—*By-pass Road.*—The Devon C.C. has been recommended to approve the proposal of the Totnes T.C. to construct a by-pass road, at an estimated cost of £20,000.

WARRINGTON.—*Sewage Disposal.*—The R.D.C. proposes to construct sewerage and sewage disposal works in the districts of Winwick, Poulton, Croft and Woolston at an estimated cost of £64,274. A further scheme is proposed to enlarge the sewage works at Burton Wood at an estimated cost of £7,847.

WILLINGTON.—*Road Improvements.*—The U.D.C. has under consideration a road improvement scheme near Burn's Farm, at an estimated cost of £20,000.

WOKING.—*Bathing Pool.*—The U.D.C. is considering a proposal to construct a bathing pool in the Constitution Hill recreation ground at an estimated cost of £13,500.

WORKINGTON.—*Swimming Bath.*—The T.C. has applied for a loan of £8,000 for the construction of a swimming bath.

Tenders Accepted.

CHICHESTER.—*Groynes.*—The M.C. has accepted the tender of A. G. Osenton, Hayling Island, for the construction of groynes at Chichester Harbour.

DONCASTER.—*Borehole.*—The C.B. has accepted the tender of the François Cementation Co., Ltd., Doncaster, at £1,474, for sinking a borehole in connection with the Thorham Farm (Cantley) water supply.

GLASGOW.—*School.*—The B.C. has accepted the tender of Wilson Bros., at £12,602, for reinforced concrete work in connection with St. Charles's School extension.

KINGSCLERE.—*Water Supply.*—The R.D.C. has accepted the tender of Reynolds & Son, Ltd., for the construction of a main and reservoir in connection with a water supply scheme for Kingsclere and Whitchurch.

LEICESTER.—*Concrete Floors, etc.*—The T.C. has accepted the tender of the Trussed Concrete Steel Co., Ltd., for reinforced concrete floors and staircases at the Corn Exchange.

LONDON (BARNES).—*Concrete Foundations.*—The tender of the Trussed Concrete Steel Co., Ltd., has been accepted for the reinforced concrete foundations for a block of maisonettes in Grove Road, Barnes.

LONDON, E.C.—*Bridges.*—The L.N.E. Railway Co. has accepted the tender of the Cleveland Bridge & Engineering Co., Ltd., Darlington, at approximately £24,000, for the reconstruction of three bridges near Fenchurch Street Station.

LONDON.—*Concrete Tank, etc.*—The tender of Walker-Weston Co., Ltd., has been accepted for the construction of a reinforced concrete tank of about 800,000 gallons capacity, and reinforced concrete foundations for three cast-iron tanks, in the London area.

SANDOWN (I. OF W.).—*Pier Reconstruction.*—The Sandown-Shanklin U.D.C. has accepted the tender of J. B. Edwards & Co. (Whyteleafe), Ltd., of London, at £22,774, for the demolition of a section of the promenade pier, and constructing in reinforced concrete the substructure of this section, and the erection of a concert pavilion.

SANDOWN (I. OF W.).—*Waterworks.*—The Sandown-Shanklin U.D.C. has accepted the tender of Aubrey Watson, Ltd., of London, at £8,891, for the construction of a sedimentation tank and ancillary work at Sandown waterworks.

SOUTHPORT.—*Waterworks.*—The C.B.C. has accepted the tender of E. Taylor & Co., of Littleborough, at £14,050, for the erection of a pump and filter house and ancillary works at Blundell House pumping station.

WEST BROMWICH.—*Sewage Disposal.*—The T.C. has accepted the tender of Tarmac, Ltd., of Ettingshall, Wolverhampton, for the construction of sewage disposal works at Ray Hall, the estimated cost being £60,000.

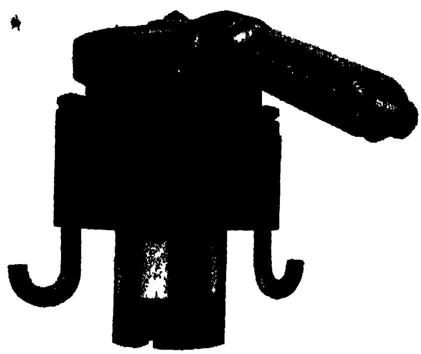
WHITBY.—*Concrete Pipes.*—The U.D.C. has accepted the tender of Fred Whitaker & Co., Ltd., Leeds, at £2,624 for laying about 150 yd. of 60-in., and 60 yd. of 39-in diameter reinforced concrete pipes, manholes, etc.

Other tenders submitted: John Parkin, £2,640; John Pearson, Ltd., £2,797; F. M. Willers & Co., £2,890; Geo. H. Graham, £2,896, R. C. Crawford & Co., Ltd., £3,222; R. A. Wilson & Sons, £3,700; William Clark & Co., £4,050.

WOKING.—*Piles.*—The U.D.C. has accepted the tender of Simplex Concrete Piles, Ltd., at £220 for driving three piles in the site for the proposed swimming bath at Constitutional Hill.

Trade Notices.

Pneumatic Vibrators.—During the last two years increasing attention has been given to mechanical methods of vibrating the formwork or the reinforcement in concrete work so as to obtain a dense consolidated concrete. When the sections are thin and heavily reinforced hand tamping is not always satisfactory and it is difficult to avoid having air holes in the finished work. For such structures as chimneys, cooling towers, and tunnel linings mechanical vibration has proved useful



Pneumatic Vibrator.

and has been extensively employed. To overcome the need for bolting vibrators to the shuttering the Consolidated Pneumatic Tool Co., Ltd., has placed on the market an air clamp vibrator. When this is used rods on the sides of the formwork are arranged to suit hooks on the vibrator, and when the throttle valve is opened the air pressure causes the hook bolts on the vibrator to grip the side rods, thus forcing the vibrator cylinder against the forms and setting up the vibrating action.

Surface Finishing Material.—A special cement is now supplied by the Cement Marketing Co., Ltd., which can be applied to concrete, brick, or stone when it is required to provide an inexpensive decorative treatment. This cement is known as "Tintocrete" and forms a durable water resisting surface. For exterior work white and light cream colours are available, while a range of bright colours may be obtained for interior decoration. In interior work Tintocrete may be applied directly to walls, thus saving the cost of plastering. Instructions for using "Tinto[illegible]te" are contained in a leaflet issued by the Cement Marketing Co. Either a paint [illegible] or a high-pressure spraying machine may be used, the proportions of the mixture [illegible]ing with the method of application.

Cast-in-situ Concrete Piles.—A brochure has been issued by the Franki Compressed Pile Co., Ltd., which contains a description of the method of driving Franki piles and their characteristic properties. Among the advantages to be gained by using the piles are stated to be (1) high carrying capacity—the normal working load is 80 to 110 tons on each pile, (2) driving to the exact depth without need for extending the pile or cutting off the head, and (3) saving in length of piles due to [illegible]esence of the enlarged base. The piles can be driven on a batter and rein[illegible] necessary. A number of illustrations of work on which the Franki pile has been used is included, together with test results. In addition to the compressed pile the Company has developed a pile that can be sunk by hand and concreted under pressure, and the "Mega" pile which is jacked down in sections where underpinning is required.

Reinforced Concrete in Slum Clearance Schemes.—At the present time when plans are being prepared in many cities for the clearance of slums particular interest is attached to descriptions of the methods adopted in other countries. Practice on the Continent differs from that in Great Britain mainly in the erection of large blocks of tenements of modern design and centrally situated instead of separate small houses on building estates often far removed from the worker's original surroundings, and more important still at a considerable distance from his work. Some large cities now contain blocks of municipal flats where workers can live close to their work and in the surroundings to which they have always been accustomed. Reinforced concrete lends itself admirably to the construction of such buildings, as can be seen from the illustrations of modern blocks of flats contained in a booklet entitled "The Rôle of Reinforced Concrete in the Development of Replanned Slum Areas," issued by the British Portland Cement Association, which contains half-tone illustrations, drawings of work recently executed on the Continent and in Dublin, and a statement of the case for the erection of blocks of flats to replan slum areas. The pamphlet can be obtained free of charge from the Association.

Recent Patent Applications.

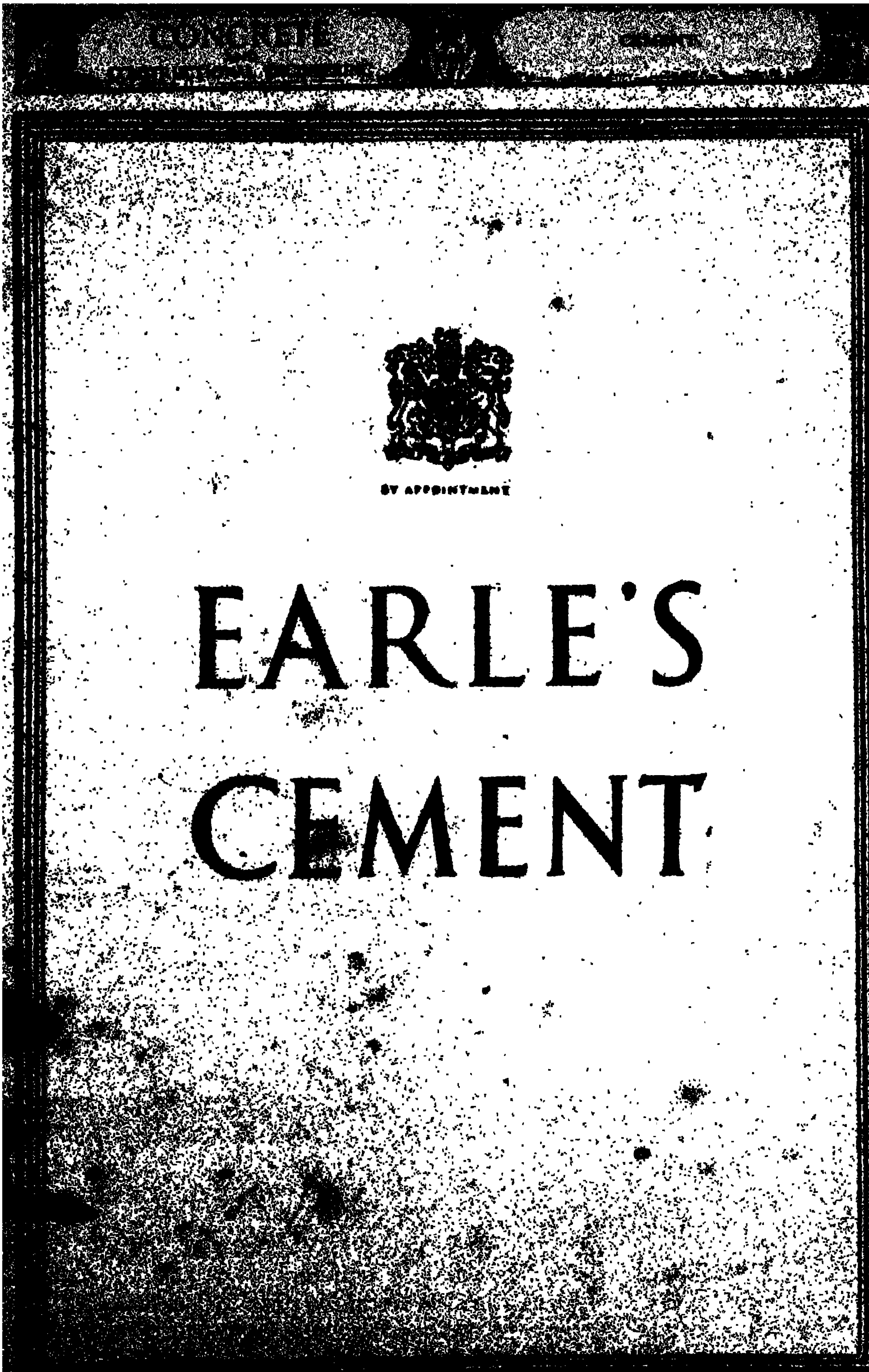
397,127.—M. N. Ridley. Reinforced concrete floors, roofs, and walls.

396,569.—F. Zollinger: Steel girders for concrete structures.

396,556.—G. A. Maunsell. Underpinning of bridges.

INDEX TO ADVERTISERS.

[*Classified Directory on p.* ii.]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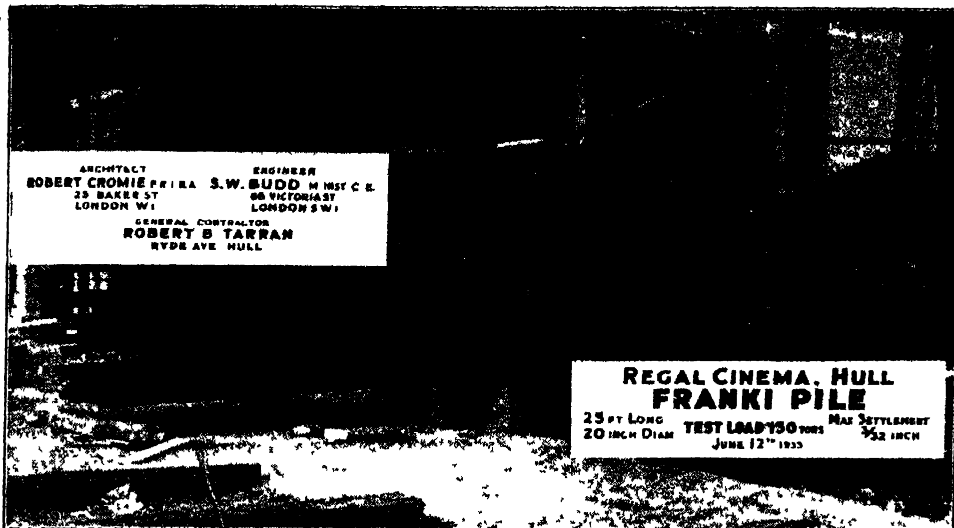

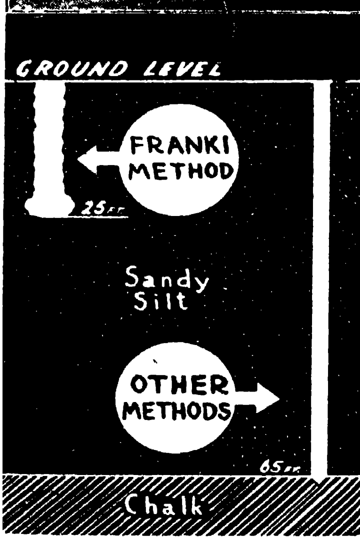

REMARKABLE TEST of Franki Piles at Hull - - -

This test pile for the Regal Cinema, Ferensway, carried a load of 130 tons with only 3/32-in settlement, which came back when the load was removed, proving that the difference in elevation was due to compression only A remarkable performance for a 25 ft pile in an area where 65 ft has been found necessary for other piles . that is, driven down to the chalk as shown in diagram Each Franki pile for the actual construction will be reckoned on a working load of 80 tons, thus coupling the economy of less and shorter piles with a big margin of safety. Architect · Robert Cromie, Esq , F R I B A , 35 Baker Street, London, W.1 ; Engineer . S. W. Budd, Esq , M Inst C.E , 68 Victoria Street, London, S W.1 , General Contractor . Robert G. Tarran, Esq , Ryde Avenue, Hull

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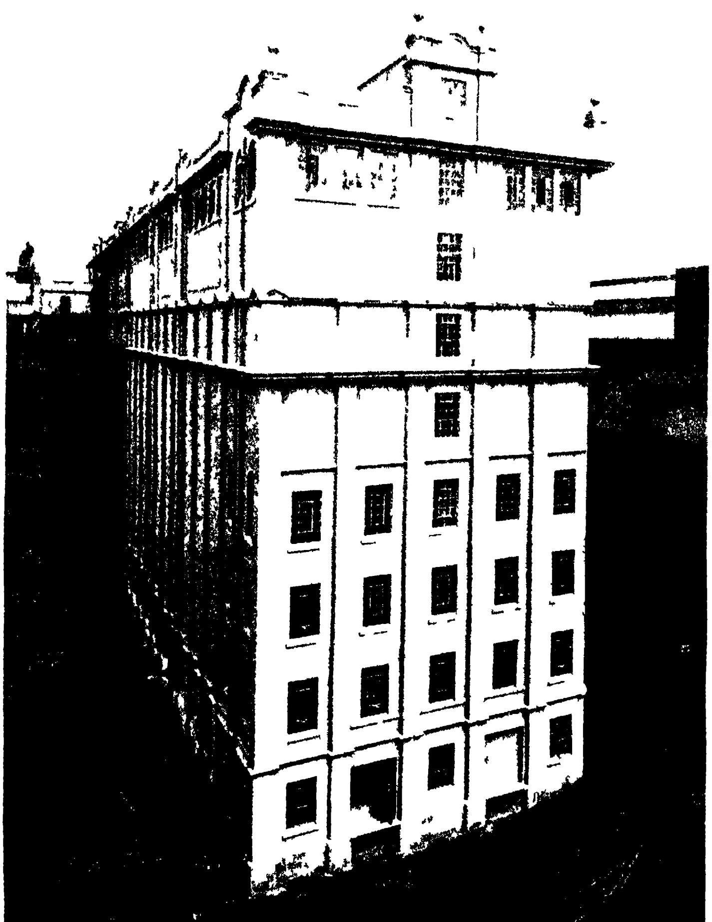

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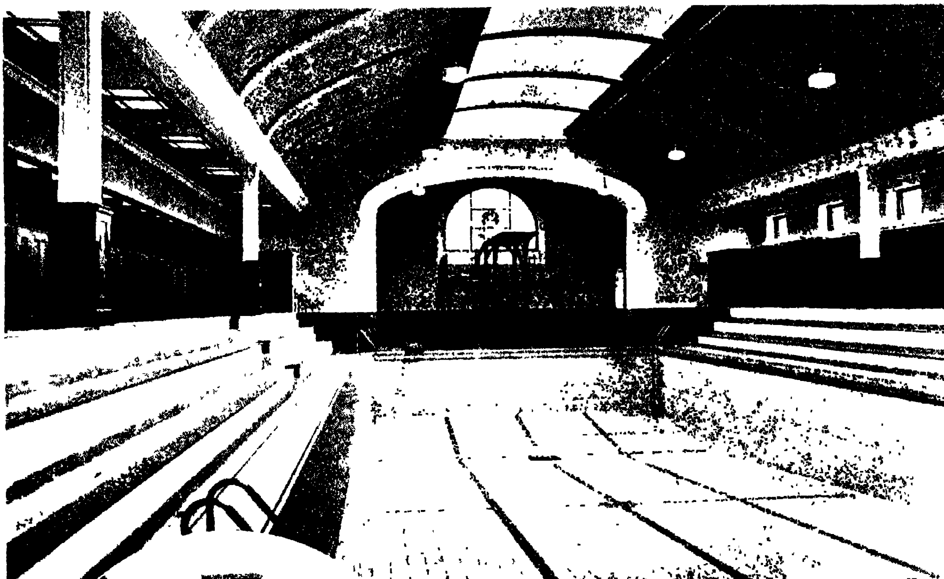

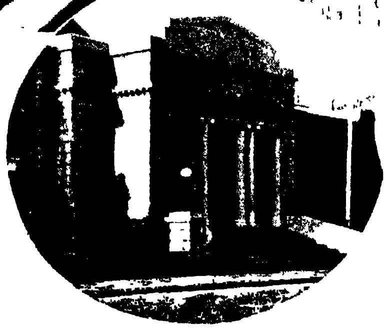

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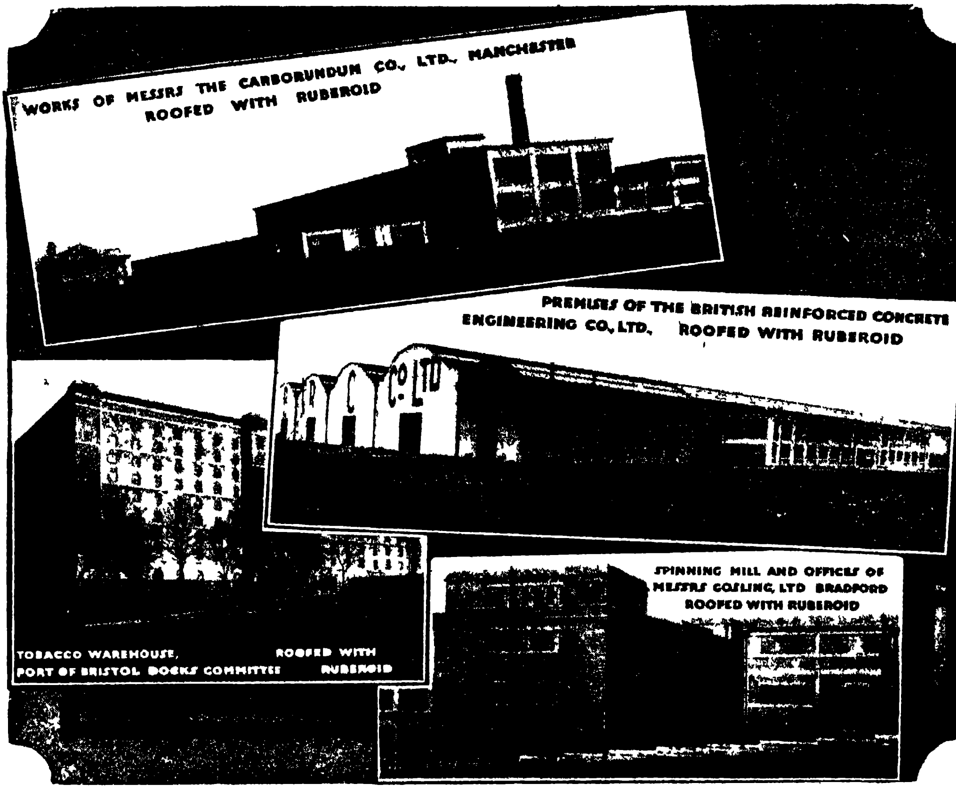

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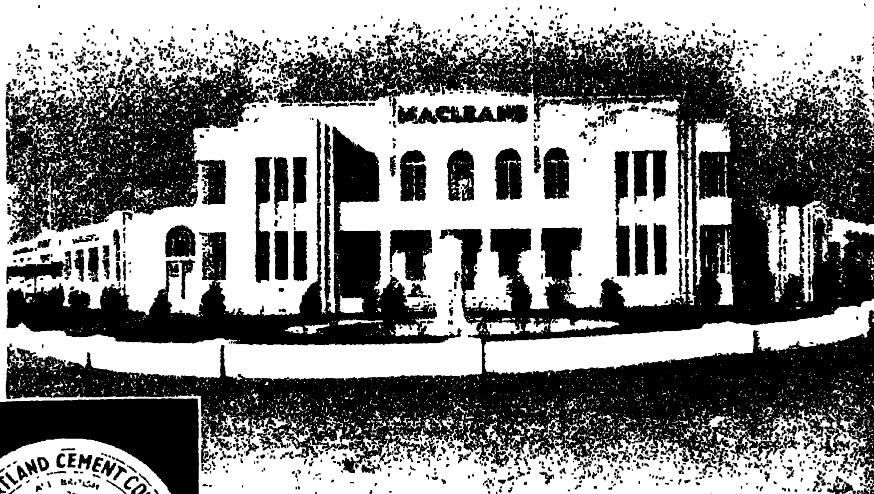

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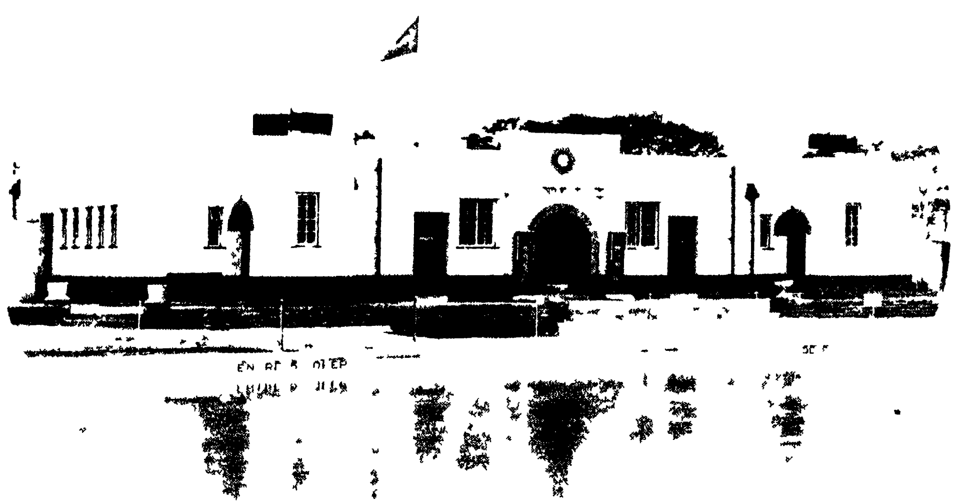

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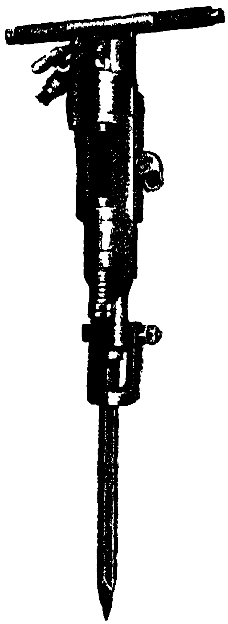

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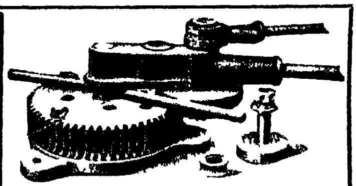

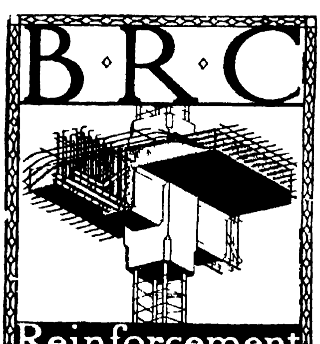

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BOOKS ON CONCRETE

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An up-to-date English book by an engineer with extensive experience in the practice of leading specialists. The Tables cover the design of every description of reinforced concrete structure, and there are fully worked examples showing how the Tables are used. Every Table is reproduced to a large size with clear lettering. Complete Examples are included of the design of beam and slab floors, flat slabs, etc., with working drawings, steel bending lists, tables of quantities, etc. If he works on the lines laid down by the author, it is possible for a junior designer to complete a design without supervision. All the short cuts and simplified methods essential for rapid solution of designing problems are clearly set out.

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The best book in the English language dealing solely and exhaustively with the subject of formwork. Gives complete designs for formwork for every type of plain and reinforced concrete structure, from simple footings to arch bridges, with tables from which the sizes and quantities of timber required for any type or size of structure can be seen at a glance.

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Estimating and Cost Keeping

"ESTIMATING AND COST KEEPING FOR CONCRETE STRUCTURES." By A. E. Wynn, B.Sc., A.M.Am.Soc.C.E. [illegible]7 pages, 9[illegible] illustrations, 2 folders. Price 15s.; by post 15s. 6d.

The author has had very wide experience in contracting for all kinds of concrete and reinforced concrete structures, and has closely studied the systems in general use. As a result of this study and practical experience the author has adopted in his own business methods of estimating, cost keeping, and book-keeping which are particularly suitable for the use of concrete constructors. The system has the advantage of extreme simplicity and ensures the achievement with a minimum of effort of the accuracy in estimating that is essential for successful contracting. Reproductions are given of all the forms required.

Encyclopædia of Concrete

"THE CONCRETE YEAR BOOK." A Handbook, Directory and Catalogue of Concrete. (Published annually on 1st January.) Edited by Oscar Faber, O.B.E., D.Sc., M.Inst.C.E., and H. L. Childe, Editor Concrete Publications Ltd. Revised every year. 1933 edition includes 780 pages. Price 3s. 6d.; by post 4s. 3d. anywhere in the world.

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"MANUFACTURE AND USES OF CONCRETE PRODUCTS AND CAST STONE." By H. L. Childe. [illegible]54 pp., 205 illus., colour plates. Price 6s.; by post 6s. 6d.

Deals exhaustively with every phase of the manufacture of pre-cast concrete, concrete products, and cast stone of every description. Selection of materials, grading and proportioning, mixing, curing, etc., are fully covered. Methods of surface treatment are described and illustrated in colour and half-tone. Moulds for all types of concrete products and all shapes of cast stone are described and illustrated by more than 100 working drawings and illustrations of moulds in wood, plaster, sand, gelatine, concrete, etc. Suitable proportions of cement and aggregate are given for all products.

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Gives complete descriptions of the methods and processes used by more than twenty manufacturers of all classes of pre-cast concrete, including the largest makers of paving flags, roofing tiles, and similar products, concrete and breeze slab and block factories, large and small cast stone works, etc., covering the whole range of pre-cast concrete units.

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CONCRETE
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Volume XXVIII No. 9 SEPTEMBER, 1933.

EDITORIAL NOTES.

The Training of a Civil Engineer.

WHEREAS engineering was at one time divided into two branches only—civil and military—there are now so many subdivisions that the training required for any one is specialised to a large degree. There is also a considerable amount of overlapping between certain branches of engineering work, and a definition which covers the duties of a civil engineer is difficult to obtain. The design and construction of railways, docks, harbours, canals, and hydro-electric works are some of the subjects with which civil engineering deals, but even among these there are some portions in which collaboration with mechanical and electrical engineers is necessary.

In this country a recognised qualification of a civil engineer is membership of the Institution of Civil Engineers; in consequence a guide to entry to the profession should be based on the requirements of this body. Those who wish to obtain the diploma of the Institution cannot do better than consult a book * which has recently been published, and whose authors are two well-known consulting engineers. The authors are strongly in favour of the pupilage system, but at the same time point out that the Institution does not make this compulsory for admission. We agree that the experience gained during a paid appointment may not be all that is desirable when considered as engineering training, but we differ from the implication in the authors' statement that "In circumstances which created the necessity for such a paid assistant, it is highly improbable that any boy would receive the personal attention from his chief which is so desirable a feature in the case of the training of a civil engineer." There may be cases in which the pupil receives the personal attention of his chief, but it is probable that these are rare except in small firms. Too often pupils are handed over to the care of assistants, and it may be that some do not see their chief from one year's end to the other. It is often forgotten that the arrangement by which a young engineer becomes a pupil is a contract under which both parties have obligations. In the majority of cases pupils are well looked after and assisted in obtaining practical knowledge, but on the other hand there are instances where engineers who cannot provide adequate opportunities for acquiring practical experience receive pupils into their offices. Cases, also, are not uncommon in which the engineer—being a paid servant of a public body—is expected to take pupils with the object of obtaining cheap labour on his staff or as a monetary recompense for the inadequacy of his own salary. At the present time there is no remedy for

* "The Training of a Civil Engineer." By C. E. Howard Humphreys and G. Howard Humphreys, MM.Inst.C.E. (London: Edward Arnold & Co. Price 5s. net.)

these abuses. The position is such that two men of equal mental ability may leave the university simultaneously and enter pupilage, paying equal premiums, and yet one may obtain valuable experience on important works and, more important still, make valuable friendships, while the second may spend a large portion of his time in plotting small surveys required for law cases, checking bills of quantities, and other such work. We do not mean to imply that the latter class of work is not of importance, but it is definitely waste of a pupil's time when it can be done by a junior draughtsman. The pupil is in the office to learn civil engineering, and in the short time at his disposal should be given an opportunity of coming into contact with as many as possible of the duties which he may ultimately have to carry out. Pupilage is not the time for exercises in arithmetic.

Outside Great Britain pupilage, whether subsequent to a university course or as a complete training by itself, is not a recognised method of entering the engineering profession. In fact we are unaware of any country where this form of training is in vogue, and there is at least one country where it is illegal. The practice in the United States, for instance, is for the young engineer leaving the university to obtain a position with an engineer with whom he obtains his practical training. In some of the Dominions the same method of entry to the profession also exists, and the young engineer is paid a salary from the beginning of his career.

The numerous discussions which have taken place on the education of engineers are evidence that there is a considerable difference of opinion regarding the correct course to be pursued, and although we have drawn attention to some of the disadvantages which are associated with the pupilage system, this has chiefly been done in the hope that some of its abuses may be removed. It is known that some engineers require the student to have taken a high place at the degree examination before they will receive him as a pupil, but there is no means by which the pupil or his parent can estimate the engineer's ability to supply the necessary practical training.

The young engineer entering upon his practical training will find much useful information in the book mentioned, of particular interest being the chapters dealing with drawing-office practice and the duties of a resident engineer in charge of a contract. Another chapter contains a discussion of the appointments which are normally open to civil engineers—at the present time there is unfortunately a surplus of applicants for these—and the salaries which are generally offered. The authors include a useful list of textbooks which " can be bought with absolute safety " ; to this " The Training of a Civil Engineer " might be added.

A Seaside Pavilion in Reinforced Concrete.

In this issue we illustrate the designs awarded the first and second prizes in the " Concrete and Constructional Engineering " competition for reinforced concrete designs in which prizes are given each year by the proprietors of this journal for the best design for a building in reinforced concrete by the atelier students at the University of London School of Architecture. The competition is set by Professor A. E. Richardson (the Head of the School), and this year the subject chosen was a seaside pavilion. The first prize was awarded to Mr. E. C. O'Farrell and the second to Mr. A. P. Ciregna. The jury consisted of Mr. Arthur Davis, F.R.I.B.A., Mr. Lovett Gill, F.R.I.B.A., Mr. H. V. Lanchester, F.R.I.B.A., and Mr. Corfiato and Mr. Collins of the Engineering Department of the University.

The Foundations of the Ford Motor Company's Power House at Dagenham.—III.

By R. V. ALLIN, M.Inst.C.E.

Manufacture of Piles.

The piles were specified to be of one part of aluminous cement to two parts of washed sand up to ¼-in. mesh with not more than 4 per cent. of silt and four parts of crushed and washed ¾-in. to ¼-in. ballast. They were cast on four 80-ft. by 32-ft. pile beds covering a total area of 160 ft. by 70 ft., allowing a space

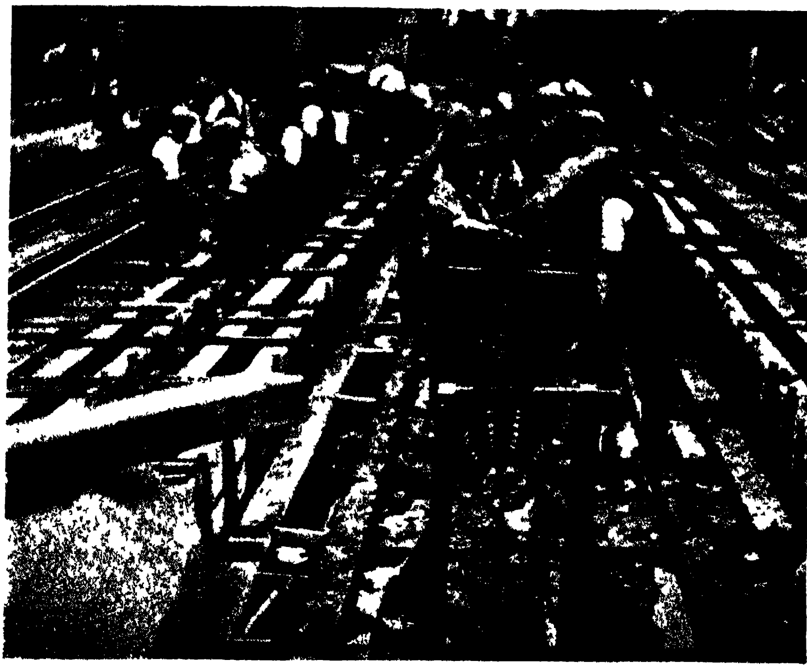


Fig. 21.

down the centre through which the mixed concrete was taken in train loads, consisting of a 24-in.-gauge petrol loco and two bottom-tipping skips, which were picked up by a 5-ton loco crane commanding the moulds.

The forms were erected on fixed timber platforms sunk into the ground and with a drained foundation. This platform consisted of 12-in. by 6-in. cross sleepers at 5-ft. centres, with 9-in. by 3-in. longitudinals spiked down to these sleepers at 2-ft. 8-in. centres, forming the ground shuttering for the bottom flats of the piles. These longitudinals were strutted laterally with 4½-in. by 2-in. stretchers. The side sections of the moulds were made in 15-ft. lengths, each side being removable when striking the moulds by slipping out the top tie bolts and slacking the folding wedges. These sections were re-used thirty-eight times during the period of manufacture of fourteen months. The rein-

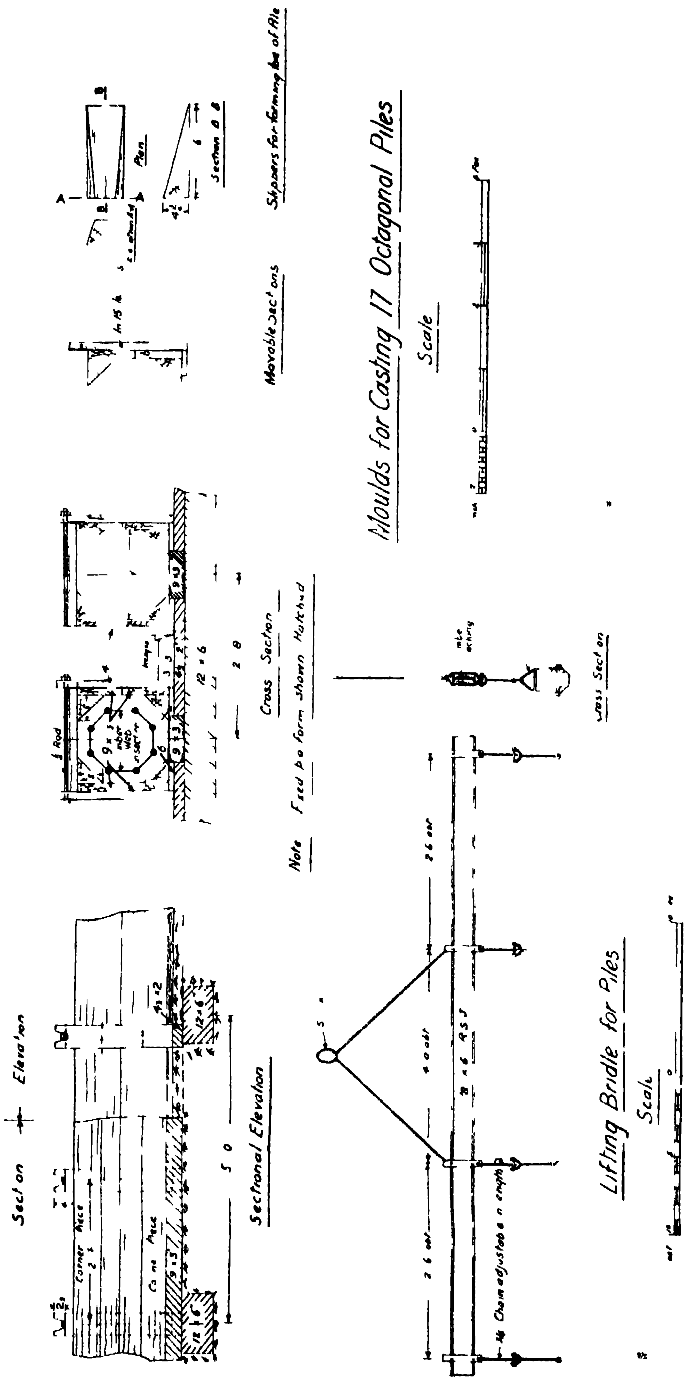

Moulds for Casting 17 Octagonal Piles
Scale
Cross Section
Note Fixed to form shown Hatched
Sectional Elevation
Section Elevation
Movable sections
Stoppers for forming toe of Pile
Plan
Section B B
Lifting Bridle for Piles
Scale
Cross Section

Fig 22

forcement was spaced with timber webs cut to take the bars and made in sections sufficiently small to allow of them being taken out of the cage before concreting (*Fig.* 22). This view also shows the lifting arrangements for four-point suspension of the piles. *Fig.* 21 shows the setting up of the reinforcement prior to fixing the moulds and the longitudinal timbers forming the bottom flats of the piles, while *Fig.* 23 illustrates the bottom-dumping skips brought up to the crane preparatory to pouring the concrete in the moulds. In *Fig.* 24 is a general view of the north side of the pile yard with the crane striking the moulds of a freshly cast pile ; this also shows the 15-ton electric derrick which picks up

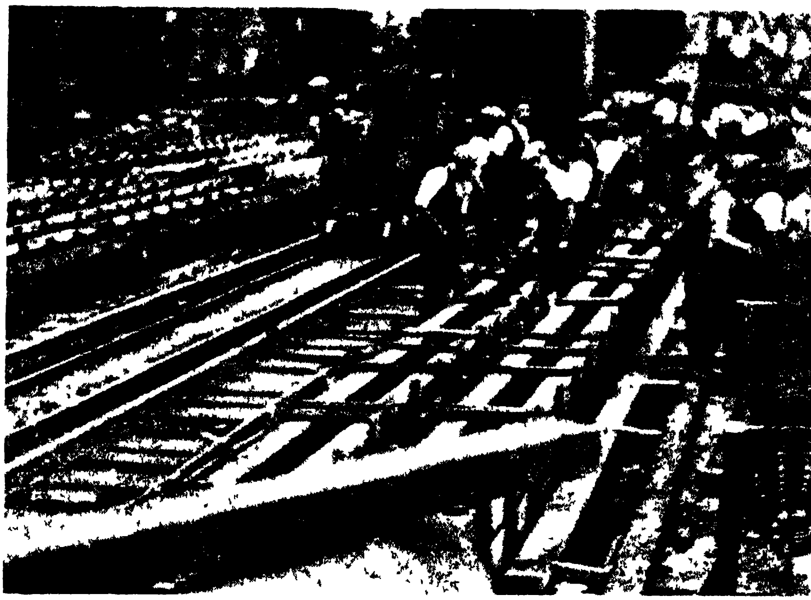


Fig. 23.

the piles from the casting beds and transports them to the stack and thence to the pile frame as required.

The sides of the moulds were struck from 4 to 6 hours after pouring, and it was specified that the piles might be moved 48 hours and driven 72 hours after pouring. It was found, however, that after a period of 60 hours the piles would usually withstand the hard driving. The piles were kept wet by sprinkling them for at least 24 hours after casting.

It was essential that the piles should be accurately cast at the head and shoe, as a head fitting tightly into the helmet led to spalling and a non-axial shoe to drift in the pile when driving. It was also necessary to ensure that no laitance should be left at the head, since this would lead to spalling in driving, and that there should be no " wind " in the length which might introduce bending stresses when driving.

The curing of the pile was of particular importance owing to the heat which developed in the setting of the aluminous cement concrete, and it was found to

have considerable influence on the strength of the piles In connection with the curing it was found necessary to drain the pile beds thoroughly, as they were laid on soft clay.

The minimum crushing strength specified for the concrete of the piles was 4,000 lb. per square inch. Daily test cubes recorded strengths ranging from 4,090 lb. to 8,280 lb. per square inch after 7 days, with an average of 6,500 lb. per square inch

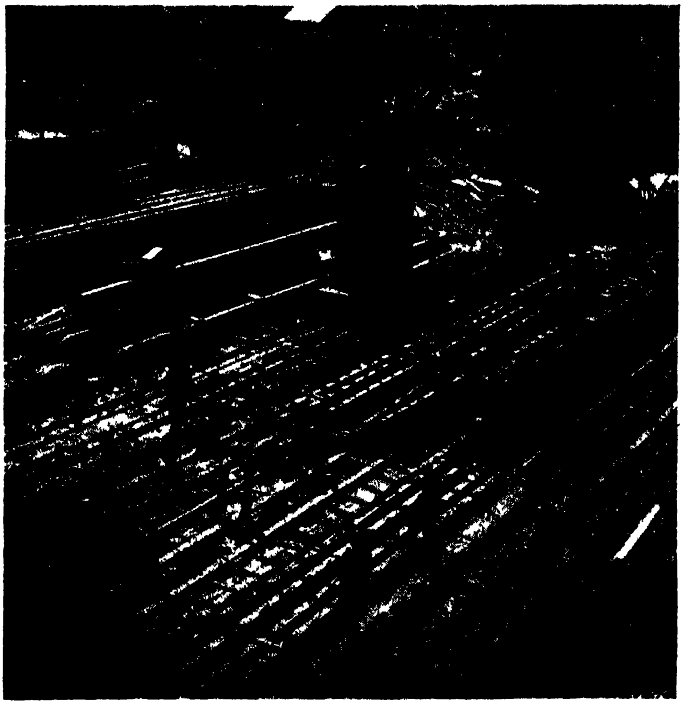


Fig. 24.

In the operations of making the piles, including unloading the aggregate at the mixer but excluding handling after casting, the following labour was employed: At the mixer station: 1 mixer driver, 6 labourers, 1 crane driver, and 1 banksman. In the pile yard—depositing and punning concrete 1 ganger, 10 labourers, 1 crane driver, and 1 banksman Setting steel skeletons: 6 steel fixers, with 1 burner and mate in attendance. Running concrete: 2 loco drivers and 2 rope runners. Repairing moulds: 2 carpenters. Watering piles: 2 boys.

The following is a list of the plant used in these operations: At the mixer station: One 3-ton steam crane and one ½-cb. yd. Fowler mixer (paraffin). At the pile yard: One 5-ton loco steam crane and one 15-ton electric derrick. Running concrete: Two petrol locomotives, 24-in. gauge.

The maximum number of piles cast in one 10½-hour net working shift was 19, representing 171 half-cubic-yard mixings.

The materials used in casting 2,045 17-in. octagonal piles were as follows:

$\frac{3}{4}$-in. to $\frac{1}{4}$-in. shingle, 7,660 cb. yd.; sand ($\frac{1}{4}$ in. down), 5,360 cb. yd.; aluminous cement, 2,240 tons; chilled cast-iron shoes (80 lb. each), 73 tons; $1\frac{1}{8}$-in. steel reinforcement, 1,790 tons; $\frac{5}{16}$-in. helical binding, 277 tons; $1\frac{1}{2}$-in. pipe ferrules (17 in. long), 14,570 lin. feet; 14- and 16-gauge binding wire, $4\frac{1}{2}$ tons. Consumable stores: Water, 860,000 gallons (approx.); steam coal, 320 tons, petrol, 2,500 gallons; paraffin, 1,235 gallons, cotton waste, 290 lb.; lubricating oil, 225 gallons, engine oil, 75 gallons; black oil, 85 gallons; oxygen, 60 cylinders; acetylene, 8 cylinders; Stauffer's grease, 200 lb.

The proportions of the aggregate used were 0·829 parts of shingle, 0·580 of sand, and 0·222 of aluminous cement. In making the piles an excess of sand was used forming a 1 : $2\frac{1}{2}$: 4 mix, as it was found by experience to give more satisfactory results. Where less sand was used the piles showed a tendency to spall. The proportions previously given include a certain amount of waste of

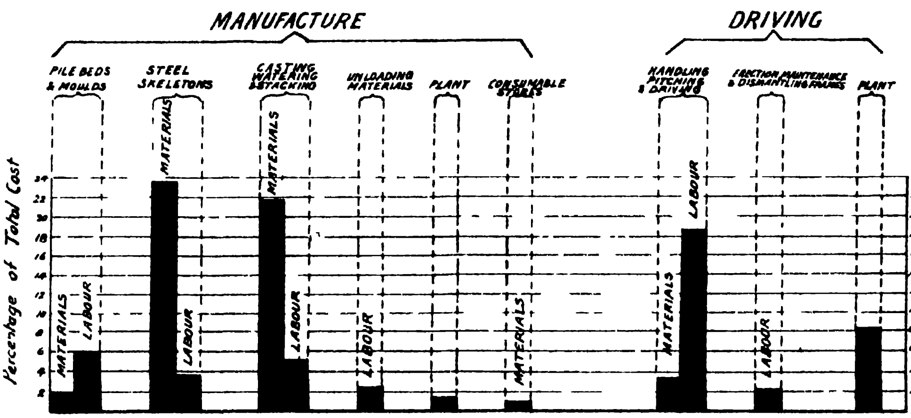


Fig. 25. Analysis of Expenditure on Piling.

the aggregate in transit. *Fig.* 25 shows an analysis of the expenditure on the manufacture and driving of the piles.

Turbine Basement.

The whole of the excavation for the turbine basement, pump house, and valve chamber was designed to be carried out within a cofferdam consisting of No. 3 Larssen steel sheet piles, driven down to -40·00 O.D. except on the west side of the pump house, where No. 5 Larssen sheeting was used, and driven to -55·00 O.D. as a safeguard against possible disturbance of the foundations there due to driving the circulating water tunnels which it was thought would reach this point after the foundations were completed. The cofferdam for the turbine basement was about 305 ft. long by 86 ft. wide at its widest point, and was divided into thirteen compartments (*Fig.* 26). Originally it was intended to carry the excavation in the open to a depth of about -28·00 O.D. in some places, and it was evident that unless special provision was made large quantities of water would enter the excavation. Heavy pumping was inadvisable owing to the close proximity of existing buildings, and it was decided to treat the ballast below the foundations within the steel sheeting by cementation.

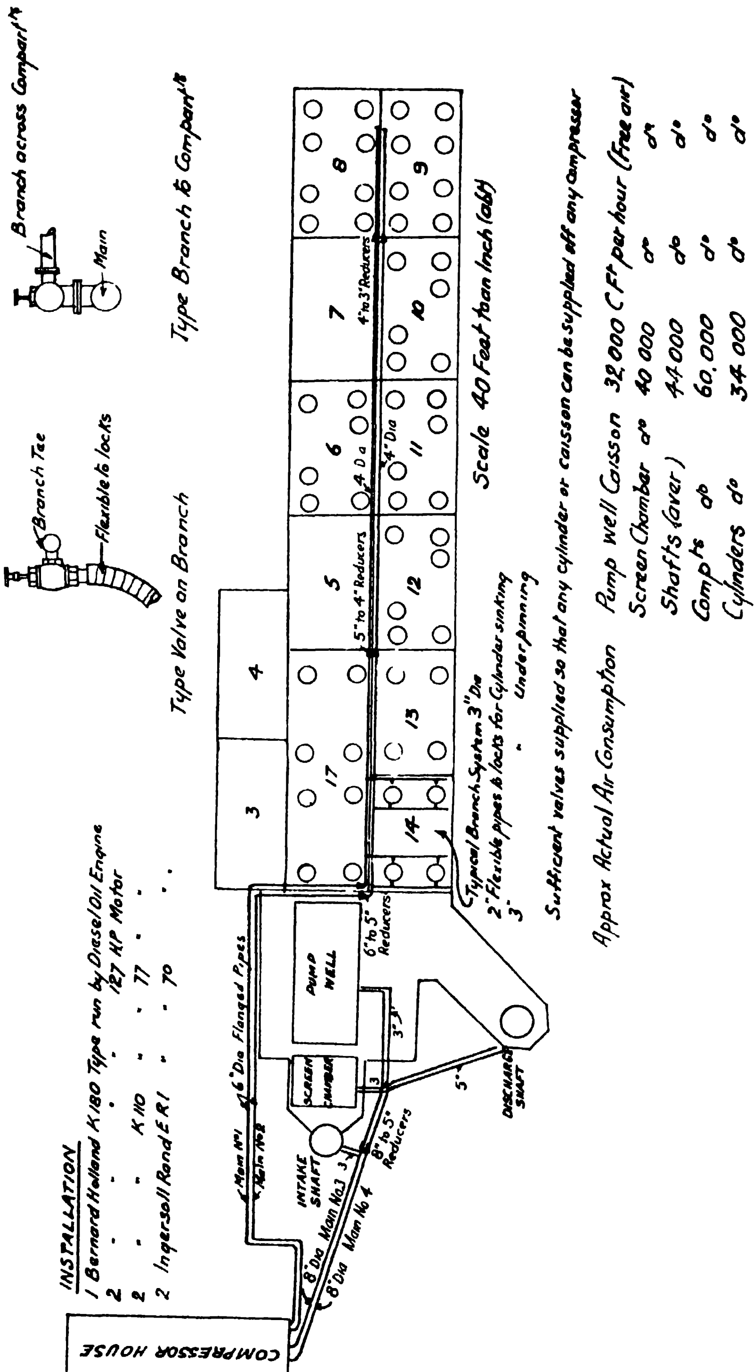


Fig 26. Compressed Air Distribution.

Cementation.

The depth and nature of the ballast and its suitability for cementation were ascertained by six core borings taken before work was commenced; in addition a number of confirmatory 2-in. flush borings were taken immediately in advance of the work as it progressed in order to explore possible local depressions of the ballast and to ensure that the cementation was not commenced at too high a level. These borings were carried out by hand and consisted of flushing (down to ballast level) 2-in. piping through which the water was passed to the shoe of the pipe. Samples of flushings were frequently taken during borings, and when a return of sand or gravel occurred it was concluded that ballast had been reached. It was proved by subsequent excavations that the observations so obtained were not very accurate and tended to show that the ballast appeared at lower levels than was actually found to be the case. It was specified that the ballast should be treated for a depth of 6 ft. below foundation levels, but this depth was later increased in places to 8 ft.

Cementation was carried out in the usual manner, hollow drill rods fitted at the bottom with perforated bits being driven with hand-operated percussive drilling machines mechanically rotated; when the bit reached the lowest required depth, the flushing water was replaced by cement-laden water which was forced under pressure through the perforations into the surrounding ballast. The cement mixture was pumped continuously during the lifting period throughout the specified depth. Alternatively, when trouble was being experienced with fast rods and leakage of cement, the usual cross bits were replaced by pointed bits which were "bumped" down instead of rotated, and cement was injected whilst drilling, commencing immediately the bit reached the top of the depth specified for treatment. These expedients were found to improve the progress of the work. The site for drilling was such as to permit of easy erection of two level working platforms. The top platform was 7 ft. above the lower, permitting the drilling to be commenced with a 10-ft. rod and carried down below the upper floor, whence it was continued and completed by the man below who used the same machine, which he then passed back to the upper level for the addition of the second rod.

The centering of the holes, which were 2 ft. 8 in. apart, was accurately done by boring 2-in. holes in 9-in. by 3-in. deals rigidly fixed over the lower platform. The "verticality" of the holes depended entirely on the care taken by the driller, as owing to the necessity of passing the drilling machine through the upper platform, satisfactory templates could not be fixed at the higher level. The average depth of ground penetrated to ballast level was 34 ft. and the average penetration into the ballast was 10 ft. 8 in. The average time taken per hole, including drilling, injection of the ballast, and the complete withdrawal of the boring rods, was 101 minutes. Each injection was carried to refusal within predetermined limits of pressure and quantity. The pressure varied, but in some instances as much as 500 lb. per square inch was developed by the pumps, but no external disturbance appears to have been incurred beyond the area treated.

The drilling and injection were carried out with seven François cementation units, and the average number in operation throughout the work was three. Each unit consisted of a cementation steam pump, a compressed-air drilling machine, and complementary equipment. The following plant was also em-

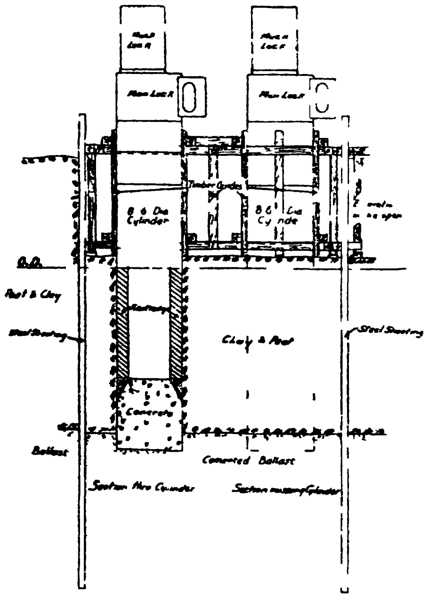


Fig. 27A.

EXCAVATION FOR SECOND FRAME AND CYLINDERS SUNK AND SEALED

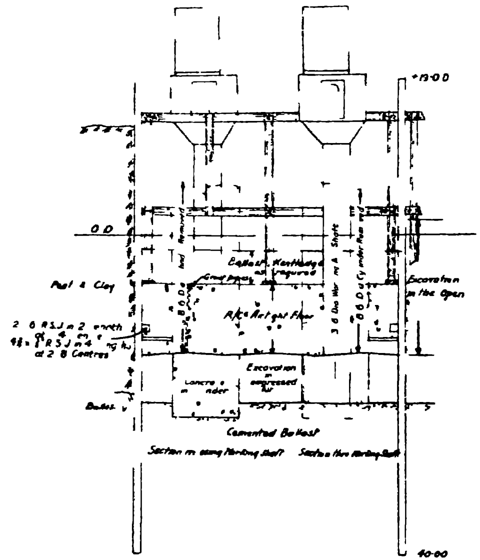


Fig 27B

CONSTRUCTION OF AIR TIGHT FLOOR AND EXCAVATION UNDERNEATH IN COMPRESSED AIR

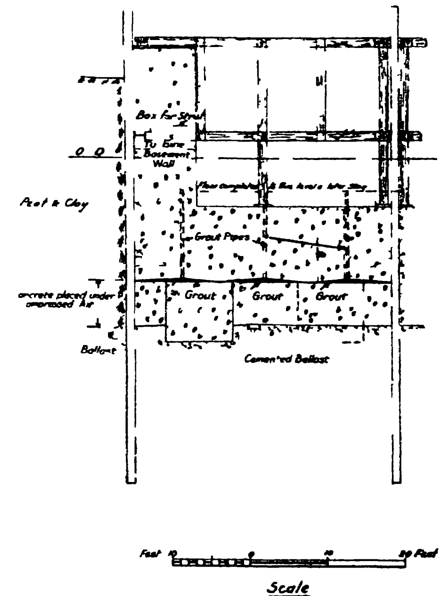


Fig. 27C.

SPACE UNDER AIR TIGHT FLOOR CONCRETED BALLAST KENTLEDGE REMOVED AND WALL BUILT UP

Construction of Typical Compartment in Turbine Basement

ployed Two oil-driven compressors of a total capacity of 380 cb ft. of air per minute at 80 lb per square inch pressure, one petrol-driven compressor with a capacity of 210 cb ft of air per minute, and two portable steam boilers The approximate quantity of the work involved is indicated by the following figures Area treated, 2,359 sq yd, volume of ballast treated, 5,240 cb yd, number of holes drilled, 3,300, total length of drilling, 147 000 lin ft quantity of cement injected, 1,405 tons, water used for all purposes 3,400 000 gallons

Excavation was commenced in compartments Nos 3, 4, 5 and 7 (*Fig* 26) It was found that compartment No 3 could be bottomed up in the dry, but in Nos 4, 5, and 7 the cementation was not sufficiently impervious to prevent the inflow of a large quantity of water accompanied by Thanet sand These compartments were allowed to fill with water, the excavation was completed, and the concrete seal placed with the aid of divers When the " blows " occurred the tide level was about 34 ft above the bottom of the excavation Owing to the vital importance of avoiding any withdrawal of the sand from the surrounding foundations previously mentioned it was considered necessary to review the design and methods of construction of this portion of the work The modification of the design has been described in Part I of these articles

Modified Construction

The position of the construction when these difficulties occurred was that in compartments Nos 6, 8, 9, 10 11 12 13, 14 and 17 (*Fig* 26) practically no work had been carried out other than their enclosure by sheet piling, sufficient excavation to fix one frame of timber, and cementation of 6 or 8 ft of the underlying ballast stratum It was decided to carry out the lower excavation in these compartments by compressed air which would completely abolish pumping The following method for completing the foundations was adopted

Excavation was carried down to allow the fixing of a second timber frame and steel cylinders of 8 ft 6 in internal diameter were sunk (partly in free air and partly in compressed air) in positions in the compartments which interfered as little as possible with the timber already in position, advantage of which was taken to guide the cylinders These cylinders were sunk by internal kentledge to a good foundation of cemented or partially cemented ballast, and the lower parts of the cylinders were filled with concrete (*Fig* 27A) The excavation in the open was then proceeded with to levels which it was judged would give a cover of about 6 ft of clay over the ballast and the cylinders removed to that level Reinforced concrete floors 8 ft to 11 ft deep were then constructed resting on the cylinders and covering the whole area of each compartment The ground shuttering for these floors was laid to longitudinal ridges and furrows with grout pipes to each ridge Under these floors the remainder of the excavation for the foundations was to be taken out under compressed air, and they were therefore provided with air shafts and locks to give access underneath. It was arranged that any surplus of upward pressure of the compressed air over the dead weight of the floors and cylinders should be balanced by placing ballast kentledge on the top of the floors (*Fig* 29) The floors were therefore only reinforced to withstand such stresses as would occur if the air pressure failed. Compressed air was then applied and the remainder of the excavation down to a good foundation, was carried out (*Fig* 27B)

In the deeper compartments the excavation was taken out in trenches about 6 ft. wide around the inside of the steel sheeting; these trenches were bottomed up and concreted in lengths and the "dumpling" was taken out afterwards. Where considered necessary the sheeting was strutted back to the cylinders, but as it had the support of the pressure of the compressed air, which varied between $\frac{3}{4}$ and $1\frac{1}{4}$ tons per square foot, the struts were only employed as a safeguard in the event of failure of the air pressure. The space formed by the excavation was then filled with concrete which made, with the air-tight

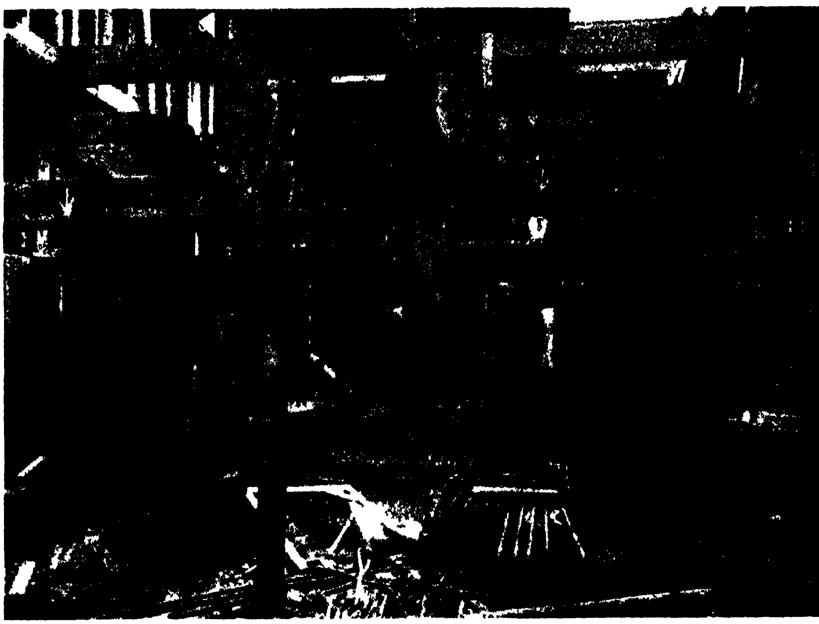


Fig. 28.

floors and the cylinders, a solid mass from the floor of the basement down to the foundation selected (*Fig.* 27c).

In all the operations of sinking and cutting off the cylinders up to the time when they received the load of the air-tight floor, enough kentledge was used to balance a possible hydrostatic pressure corresponding to the highest recorded tide level of + 17·30 O.D. (*Fig.* 28) and the cylinders were founded at levels ranging between 20·00 O.D. and - 39·00 O.D.

Fifty-six cylinders were sunk in the nine compartments and the whole of the work was carried out without any incident calling for comment. In compartments Nos. 6 to 11 at the east end of the power house there were abundant evidences of the old breach referred to in Part 1 of these articles, and a pre-Elizabethan anchor was discovered at about - 25·00 O.D. (31 ft. below shore level). For this reason it was decided to subject the cylinders in these compartments to a bearing pressure test by loading them with kentledge after they had been sealed and before the surrounding excavation had began. In addition to the application of sufficient weight to overcome the estimated skin friction

of the surrounding material (2 cwt per square foot) the cylinders were loaded to give a pressure on the foundations of 5 cwt per square foot The resulting settlements varied from nil to 1¼ in with an average of $\frac{5}{16}$ in The settlements recorded for cylinders founded at higher levels tended to be less than for those

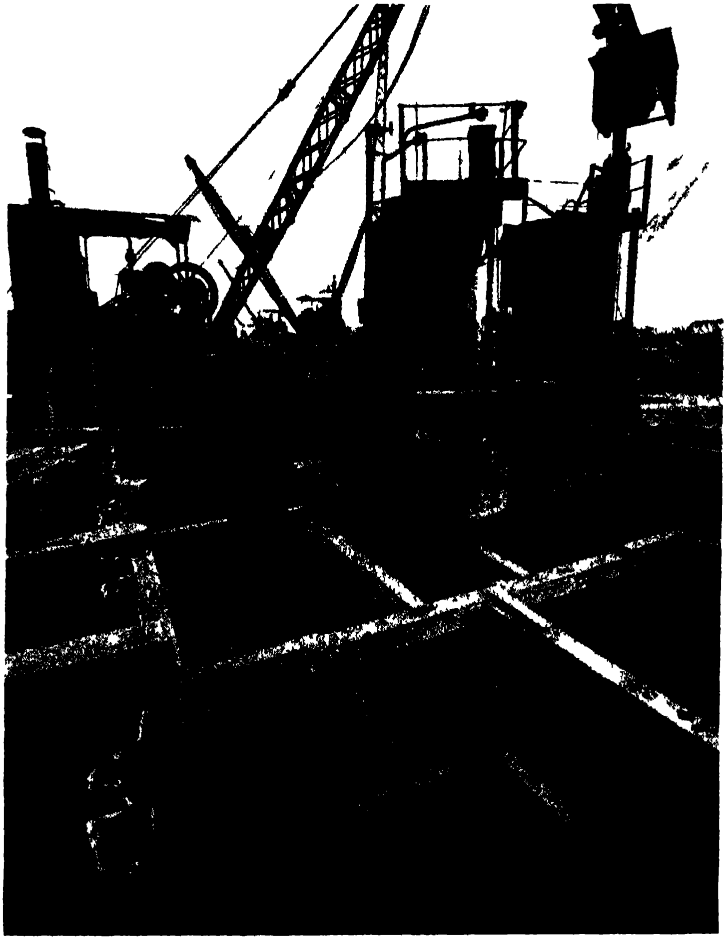


Fig 29.

founded at lower levels, due possibly in some measure to the containing effect of the sheet piling

Air-tight Floor.

The lowest sections of the cylinders which were sealed with concrete were provided with splice bars to connect them to the air-tight floors These floors were reinforced to withstand only their own dead weight and that of the ballast kentledge referred to The amount of air pressure allowed for in providing ballast kentledge on the top of the air-tight floors was judged from the pressures

found necessary in sinking the cylinders, and varied between 12 and 19 lb. per square inch.

The floors were poured in lifts. The first lift was 4 ft. high and contained all the reinforcement. At a distance of about 2 ft. above the underside of these floors lengths of 3¾-in. by 1¾-in. rolled steel joists were welded on the Larssen steel piling at about 2 ft. 8 in. and at centres projecting 4 ft. into the concrete. The object of fixing these lugs was to check the tendency for a gap to be formed between the concrete and the sheeting (caused by either air pressure or shrinkage of concrete) which would have resulted in excessive loss of air. The lugs were found to be effective, as the leakage of air was negligible.

In compartments Nos. 6, 8, 9, 10, and 11, in order to distribute some of the weight of the foundations on the sheet piling, 12-in. lengths of 10-in. by 6-in. rolled steel joists about 1 ft. 4 in. apart were welded to the sheeting 3 ft. above the underside of the floors (*Fig.* 7).

The depth of the excavation under the floors varied from 7 ft. to 16 ft., and care had to be taken that the bottoms of the cylinders were not undercut. The excavation was carried out so that at no point should a line drawn from the excavation to the bottom of any cylinder have a steeper up-grade than one in ten. In the deeper compartments rolled steel joists were left under the shuttering of the air-tight floors before they were concreted, and subsequently used as walings against which the steel sheeting was strutted back to the cylinders. When filling with concrete the space under the floors and up to the roof, the bottom of the first lift was put in fairly wet to fill the inequalities in the ballast bottom. In packing up to the roof of the chamber the concrete was mixed stiff, but owing to the settlement a space was formed below the soffit. Grout was poured in the first instance down the working air shafts and had therefore a pressure due to any head to which it may have risen in the shaft in addition to the air pressure. Enough time was left for shrinkage of the cement and then a second pressure grouting was applied through the pipes in the ridges of the roof (*Fig.* 27c).

(*To be continued.*)

International Road Congress.

The next congress of the Permanent International Association of Road Congresses is to be held in Munich in September 1934. The address of the British Organising Centre is 7 Whitehall Gardens, London, S.W.1. Papers are to be read by Mr. H. E. Lunn, County Surveyor of East Sussex, Mr. J. E. Swindlehurst, Borough Engineer of Hampstead, and Mr G H Whitaker, City Engineer of Cardiff, on progress in concrete roads; by Mr. H T. Chapman, County Engineer of Kent, Mr. A. T. Gooseman, City Engineer of Leicester, and Mr. F. W. Smart, County Surveyor of Bedfordshire, on progress in the use of tar for road work; by Mr. D. Edwards, Borough Engineer of Brighton, Mr. A. C Hughes, County Surveyor of Hampshire, and Mr. E. J. Stead, County Surveyor of Somerset, on the progress in the use of bitumen for road work; by Mr. G. S Barry, County Surveyor of Ayrshire, Mr. S. G. Stanton, Borough Engineer of Southampton, and Mr J. P. Wakeford, Borough Engineer of Stockton-on-Tees, on the use of emulsions; by Mr. O. Cattlin, Borough Engineer of Lambeth, Mr E. H. Collcut, County Surveyor of Cornwall, and Mr. T. Somers, City Engineer of Glasgow, on road surfaces; by Mr. H. H. Humphries, City Engineer of Birmingham, and Mr A. J. Lyddon, Ministry of Transport, on the relationship between road surfaces and traffic; and by Mr. G. E. Ashforth, County Surveyor of Cheshire, Mr. C. L. Howard Humphreys, and Mr. E. S. Perrin, Ministry of Transport, on regulations governing road vehicles Papers are also to be read on the safety of traffic.

Railway Viaducts near Belfast.

MAIN line trains of the L.M & S.R (N.C.C.) from Belfast to Londonderry and Portrush have to reverse at Greenisland, about 6¾ miles from Belfast, thus involving delay to the trains and additional expense, owing to the necessity of maintaining the necessary staff. With a view to providing an improved service to Portrush and securing substantial economies it was decided to construct a loop line whereby trains can run straight through from Belfast to the north This work is being carried out by the railway company in collaboration with the Government of Northern Ireland as an unemployment relief scheme.

The new line is about 2½ miles long, and crosses several roads and numerous streams. All bridges over and under the line are being carried out in reinforced concrete. At a point about three-quarters of a mile beyond the commencement of the new loop line the existing railway crosses a deep glen by means of a two-span masonry arch. The new loop line is about 200 ft. farther upstream and, crossing at an angle, requires a viaduct about 630 ft. long. At the Belfast end of this structure the line to Larne passes underneath that to Londonderry and crosses the glen on a line approximately parallel to the existing structure. The second viaduct, known as the down-shore viaduct, is about 400 ft long, and the up-shore line is carried on the existing masonry arch The undercrossing at the Belfast end of the main line viaduct has a skew angle of 16 deg., and is also constructed in reinforced concrete

Both the down-shore and main-line viaducts are similar except as regards the width, the number of arches, and the height of the piers The former consists of one main open-spandrel parabolic arch with a clear span of 89 ft. and a 30-ft rise; this is flanked on each side by three small filled-spandrel arches of 35 ft clear span and 11 ft 0 in. rise. The main-line viaduct has three similar large arches with five small arches at the Londonderry end and two small ones at the Belfast end which connect with the undercrossing General views of the viaducts are shown in *Figs.* 1 and 2.

Fig. 1.—General View of Viaducts.

The down-shore viaduct, which carries a single pair of rails, is straight; the rail level is horizontal and is about 40 ft. above the level of the stream. The arches are 17 ft. wide. The main piers are carried down to a foundation of sand-

stone, while the piers of the small arches are supported on separate footings on boulder clay. Considerable difficulty was experienced in excavating for these piers as they are in the side of the existing railway bank, and it was necessary to maintain traffic while work was in progress. The piers are 6 ft thick at the springings, have a batter of 1 in 60, and are heavily reinforced to withstand the large moment induced by unequal live loading

for each arch parallel the piers are trapezoidal. The large piers are built of mass concrete and are supported on spread footings 35 ft. by 39 ft. resting on boulder clay overlying firm sand. The piers at each end of the large spans are constructed as abutment piers.

The arch ring of the 89-ft. arch is 2 ft. 6 in. thick at the crown and 5 ft. thick at the springing, and is reinforced with $1\frac{3}{8}$-in. diameter bars at 6-in. centres at the springing and $1\frac{1}{8}$-in. bars at similar

Fig. 2.—Plant View.

The main-line viaduct is on a 60-chain curve and has a rising gradient of 1 in 75. The large arches have a maximum height of 70 ft. above stream level. This viaduct carries a double line, and the arches are 29 ft. wide. The small piers are similar to those on the down-shore viaduct, except that they are much higher and much more heavily reinforced. They are supported on spread footings 22 ft. by 35 ft. carried down to good boulder clay. As this viaduct is on a curve and it was desirable to have the opposite springings

centres at the crown The cross-walls are 2 ft thick and reinforced with $\frac{3}{4}$-in. bars at 12-in. centres. These walls are spaced at 12-ft. centres and carry a floor slab 14 in. thick reinforced with $\frac{3}{4}$-in. bars at 8-in centres. Sliding bearings are provided where the floor slab rests on the piers. The 35-ft. arches are 18 in. thick at the crown and 2 ft. 6 in. thick at the springing, and are reinforced with $1\frac{1}{8}$-in. bars at 6-in. centres at the springing and $\frac{7}{8}$-in. bars at the crown.

The spandrel walls have a maximum

thickness of 2 ft. 6 in. at the springing and a uniform batter to 1 ft at rail level. Some mass concrete filling is provided over the tops of the piers, while the remainder of the filling is rough stone and ballast. A 2-ft. thickness of stone ballast is provided over the deck of the main arches.

Both structures have been designed to carry the British Standard Load for Railway Bridges (18 units) with the standard impact allowance. The working stress in the concrete is 750 lb. per square inch in compression and 75 lb. per square inch in shear. The reinforcement is round mild steel to British Standard Specification No. 15, and was designed for a working stress of 18,000 lb per square inch.

Concrete Placing.

The total quantity of concrete involved is about 17,000 cb. yd. and, as it is spread over a considerable area, close attention was given to the arrangement of the plant. Advantage was taken of the existing viaduct, and a three-compartment bin was built spanning the stream at this point so that materials could be brought in by rail and unloaded directly into the bins. The bins are fitted with volume batchers leading into a hopper which in turn can be arranged to feed either of two 10-cb. ft mixers (*Fig* 3). The cement, which is rapid-hardening, is delivered in paper sacks and unloaded into a cement shed at rail level Two bags of cement are required to each batch, and the material is conveyed to the hopper through an inclined pipe carried down the back of the bins. The mixer operator controls the admission of the aggregates from the mixing platform. The cement batch is handled by men in the cement shed who release the batches in accordance with signals from the mixer operator The concrete is mixed for a minimum time of one minute, which is controlled by a batch meter, and is discharged into side-tipping trucks which

Fig. 3.—Central Mixing Station, showing Bins, Cement Shed and Mixers.

are taken to the placing masts. Water is pumped from the stream to a storage tank, and from there is fed to a service tank supplying the tanks on the mixers. These tanks are set for the requisite quantity for each batch, and the setting is only altered on the instruction of the engineer.

The down-shore viaduct was constructed first, and a ½-cb. yd. mast placing plant covered the whole of the work from two positions. In the main line viaduct three masts (*Fig.* 4, pp. 528–9) were used, a heavy mast 160 ft. high at the centre and two lighter masts about 100 ft. high at each end. A Blondin wire was suspended across the tops of these masts and used for slinging lines of chutes (*Figs.* 2 and 5). The concrete was conveyed from the central mixing plant to the centre and one of the end masts by means of power-operated inclined tracks. It was not possible to feed the other end mast by this method, and advantage was taken of an existing siding to deliver materials at this point. These were batched by hand and fed through another 10-cb. ft. mixer which discharged directly into the mast bucket. When the Londonderry end had been completed this mast was removed and re-erected at the Belfast end where it assisted in placing the concrete in the crossover. In this case also it was not possible to use the central mixing plant and the same arrangement was adopted as before. With the exception of this mast, the winch of which was steam driven, and the mixer, which was petrol driven, all the plant on the work was operated by electricity. Power was first obtained from plant installed on the site, but when the demand exceeded the capacity of this plant a supply was obtained from the Belfast Corporation.

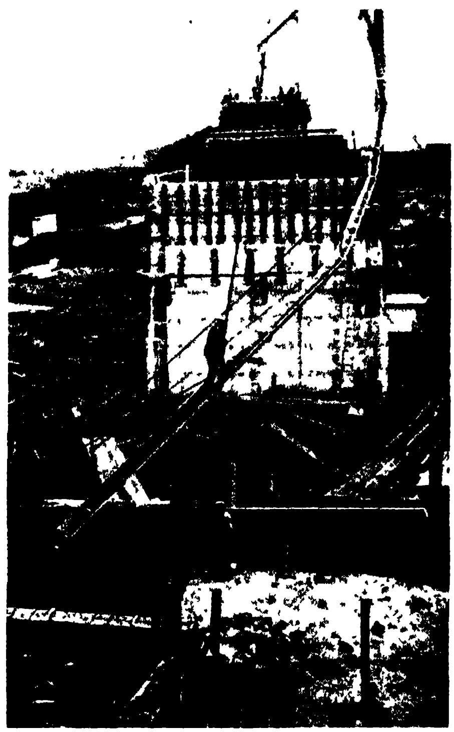


Fig. 5.- Looking North along Main Line showing Main Piers and Long Chute Line, suspended from Blondin Wire.

Where placing conditions were fairly

easy it was possible to handle 10 cb yd hourly, but when much ramming was required the output was reduced to about 6 cb. yd hourly per mast The total average weekly output was 500 cb yd The longest line of chutes used was 230 ft, and it was found that concrete with a slump of 2½ in. could be placed effectively.

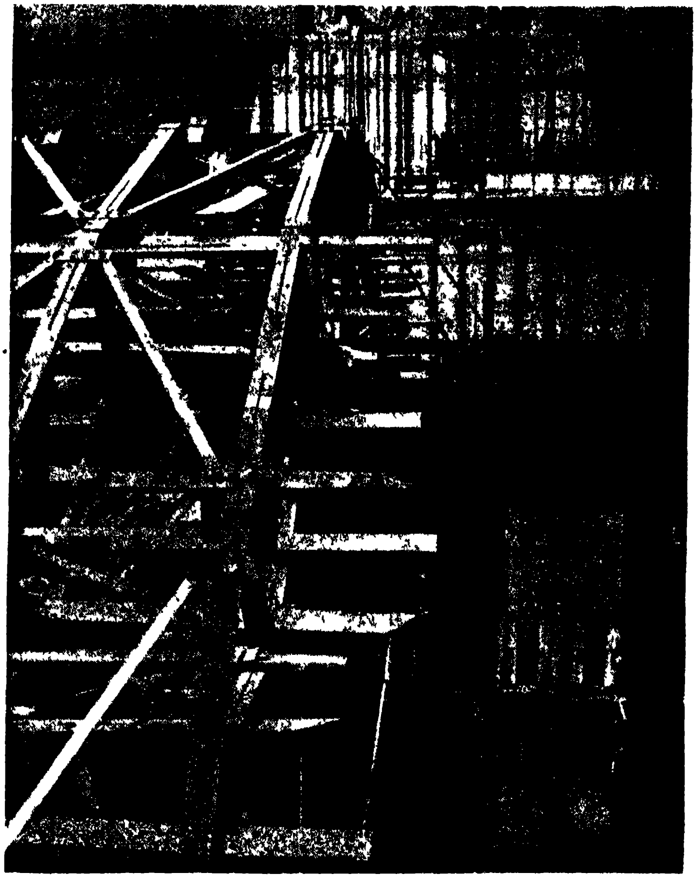


Fig. 6.—Arch Centres, Main Line, 89-ft. span.

Surface Treatment.

As no money was available for special treatment of the concrete surfaces the most suitable way of avoiding a monotonous effect on these large areas became a matter of importance, and it was decided to leave the concrete as it came from the forms, with board marks visible. On the larger piers the boards were 8 in wide and on the smaller 6 in wide, horizontal chases were inserted at 4-ft. intervals Advantage was taken of this to form the construction joints at these points, and the result is a structure singularly free from defects The piers were too irregular to make it economical to use standard steel forms, and as these would have had to be lined to give the effect desired timber was used throughout Climbing forms were designed which enabled one set of formwork to be available for the full height of each pier These forms consisted of 1½-in. tongued-and-grooved sheeting made into panels 4 ft. high and of convenient length. Double 7-in. by 2-in. studs were bolted

to the face of the previous lift and projected a distance of 4 ft The panels were fastened to these studs, and ⅝-in rods threaded through strawboard tubes and screwed into 3 in by ⅝-in square plates were fixed to the top end of the studs These were concreted in and used for securing the studs when moved up to the next position The rods were then screwed out and the holes plugged with cement mortar By this method it was hinge at the crown and 1¾-in tie rods The shape of the arch was obtained by fixing timber nailing strips to the back of the trusses The lagging was 6-in by 3 in in section The spandrel wall forms, which had horizontal sheeting, were completed before the arch was cast and the portion in contact with the face of the arch was lined with ⅛-in Masonite Tempered Presdwood so as to provide a smooth face free from form marks

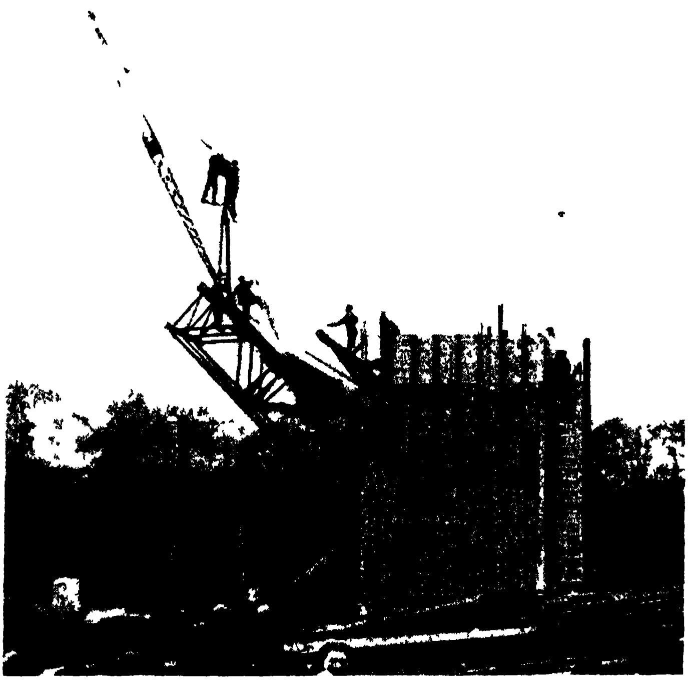


Fig. 7. Erection of 89-ft. Arch Centres on Main Line. Crane in position on nose of First Section.

possible to strip and re-erect a complete lift of formwork in one day As many as fifteen uses were obtained from one set of forms

Centering.

As the arches were a considerable distance above ground level it was found economical and much quicker to use steel centres supported from the piers rather than to build up the usual type of timber trestle from the ground In the small arches these centres consisted of steel trusses at 5-ft centres provided with a

The large arch centres (*Figs* 6 to 9) were also three-hinged trusses with 2⅛-in diameter tie rods The trusses were spaced at 5-ft centres and both top and bottom chords were braced, while the 6-in by 4-in lagging was secured to nailing strips as in the case of the small arches In the case of the down-shore viaduct the piers were erected to their full height before the arch was commenced and the centres were assembled on the ground and rotated into place about the lower hinge by means of tackle fastened to the top of the pier. Each

centre had a reaction of about 60 tons and was carried on cast-iron wedges. Extensions were provided to the bearing shoes, and the arch was struck by placing jacks under these extensions and transferring the load from the wedges to the jacks The wedges were released by hand and the centres lowered on the jacks. The trusses were then moved sideways and dismantled by tackle slung from a cantilever beam fixed at the crown of the arch In the case of the main-line viaduct the arches were cast in halves, three centres being used for one half, when this was completed the centres were moved sideways into position for the second half In this case the piers were not completed before the erection of the viaduct, and were jacked across a distance of 14 ft. 6 in. into their new position. They were then levelled, and the second half of the arch ring cast. On completion of this half the centres were lowered to the ground by tackle suspended from the underside of the arch ring.

In the down-shore viaduct the small arches were poured in one day and the centres struck in four days. On the main line the small arches were poured in two successive days and the centres struck on the fourth day, after the last section had been cast In the large arches of both viaducts the arches were cast in five sections, poured on successive days, and the centres struck on the fourth day after the last section had been cast. The

Fig. 8. -Closing Centres of 89-ft. Arch on Main Line.

centres, and these were placed by a cantilever method A small jib crane was mounted on top of the pier and placed the columns and beams and the bottom sections of the centres These were tied back to the pier, and the crane was then mounted on the nose of the middle centre and the remaining sections of the centres lifted into place This process was repeated for the other half of the centres, and the crane removed The halves were thus in position, tied back to the piers, but were within 18 in of meeting at the crown. The tie-backs were then eased until the crown hinges entered, tie rods were adjusted, the centres levelled on their wedges, and the lagging placed. The centres were struck by the same method as that used in the down-shore maximum deflection at the centre of the large arches the day the centres were struck was ⅛-in

Forms for the cross-walls and floor slab were made in panels similar to the formwork for the piers, and presented no difficulties

The intersection has a square span of 15 ft. and an average skew span of 65 ft The roof is a slab 15 in deep reinforced with ⅞-in rods at 9-in. centres spanning between cross-beams at 12 ft centres. These cross-beams have a clear span of 15 ft. and are placed at right angles to the abutment. They have an effective depth of 3 ft 3 in., a stem width of 2 ft., and are reinforced with eleven 1-in. bars. Owing to the extreme skew of this crossing it was necessary to provide a parapet

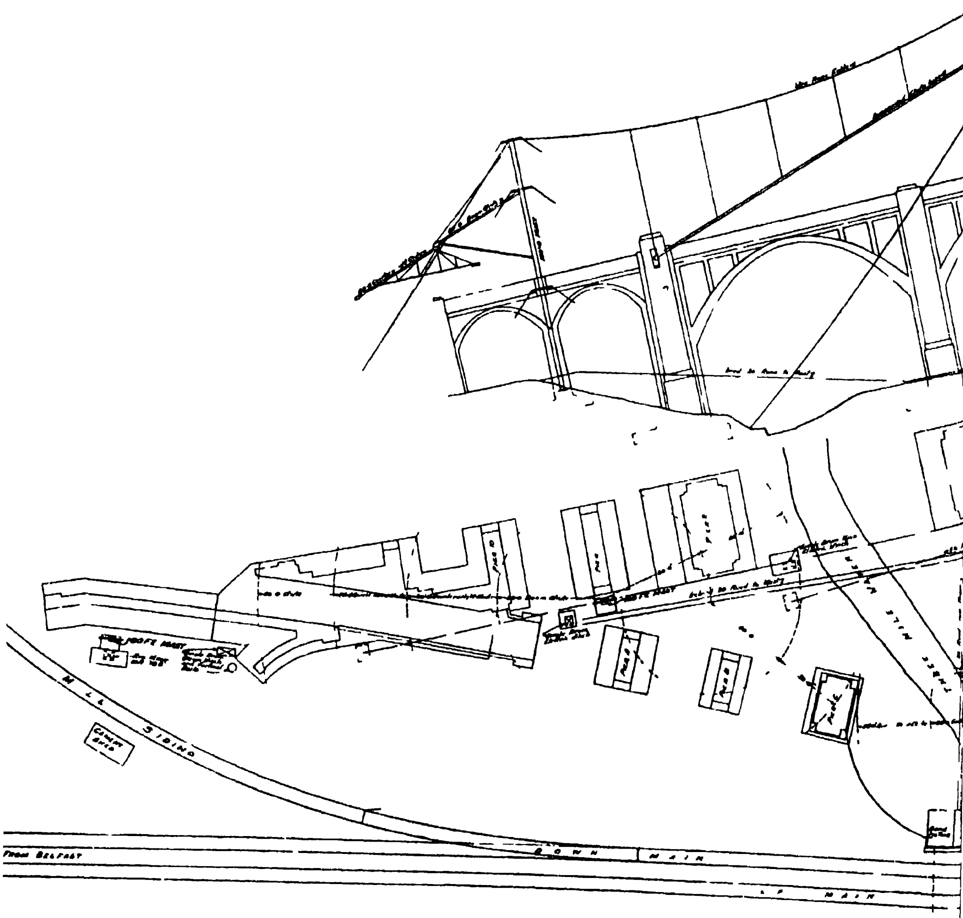
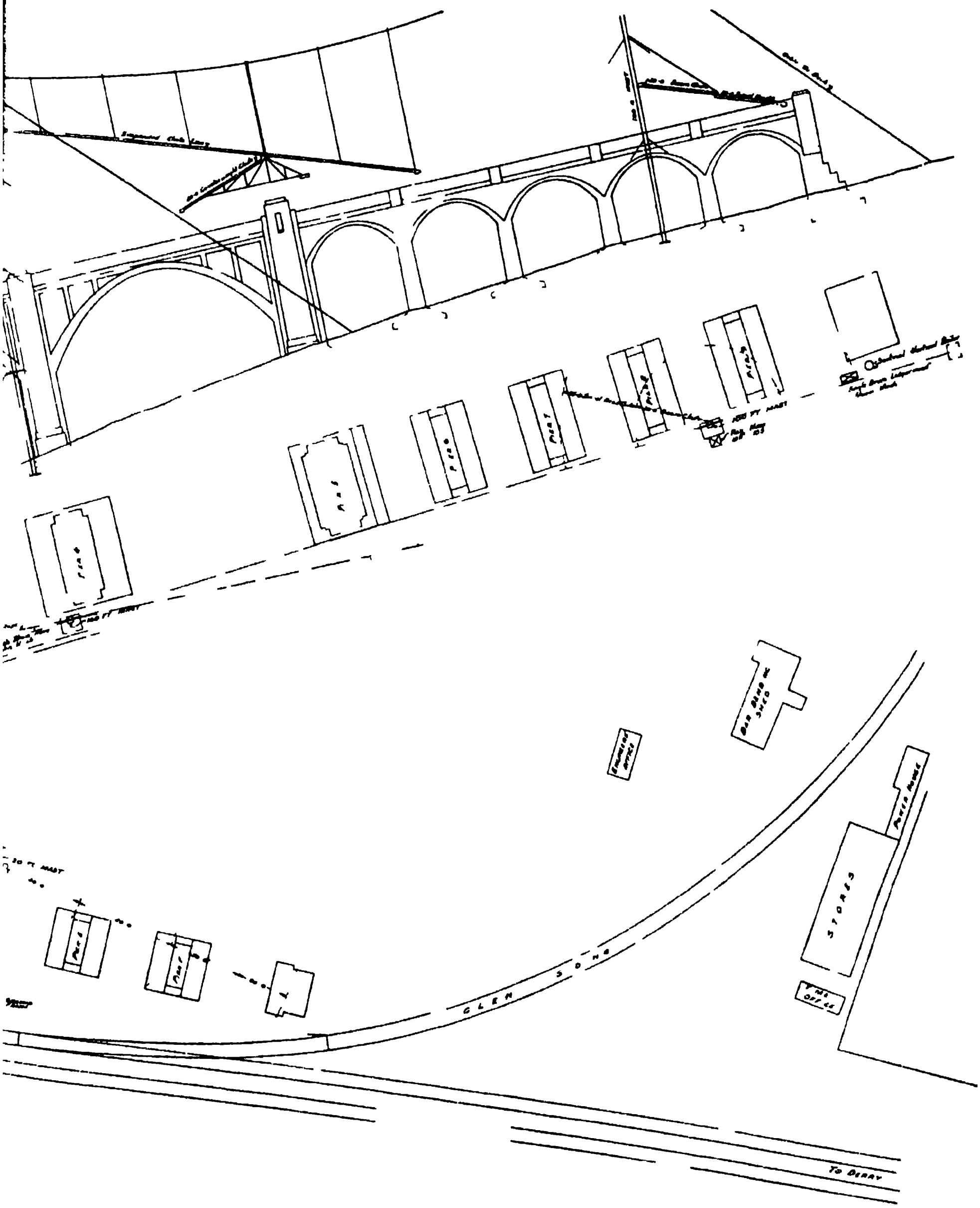


Fig. 4. Gen



t of Plant.

beam to carry the outer ends of some of the cross beams This beam is of reinforced concrete 10 ft 10 in deep by 20 in wide and is reinforced with eighteen 1¼ in bars One end of these parapet girders was fixed, and the other end was carried on a rocker bearing The abutment walls were constructed in 4 ft lifts with climbing forms similar to those used in the piers The formwork for the floor slab was carried by rolled steel joists set in pockets on the abutment wall

On the Belfast side of the crossover there is a counterforted retaining wall 165 ft long with a maximum height of about 29 ft above foundation level As there are new embankments on each side of this wall the lower part is slotted The counterforts are 18 in wide and spaced at 12 ft centres, and the curtain wall ranges in thickness from 15 in to 9 in An expansion joint is provided at the centre of the wall

Fig. 9 Main-Line, 89-ft Arch Centres moved to concrete second half of Arch.

Concrete Proportions.

All concrete was made with one part of cement to 2¼ parts of sand and 3¾ parts of basalt screenings graded from ⅜ in to ¾ in As it was impossible to secure washed sand at a reasonable price a washing machine with an output of 40 tons a day was installed Considerable attention was devoted to the production of the concrete and the maximum slump permissible was 2½ in It was found that this enabled the material to flow easily in the chutes and a dense concrete with a good face was secured Six concrete cubes were secured for each day's run of each mixer and these were stored in a special shed with thermostatically-controlled heating Notwithstanding these precautions it was found that owing to variations in the aggregate it was not possible to secure a concrete of uniform strength The specification required a breaking strength of 2,000 lb per square inch at seven days and 3,000 lb at 28 days These strengths were secured, but the range of the values was considerable All the reinforcing steel, of which 700 tons were used, was bent on the site by a power machine

The whole of the work is being carried out by the railway company by direct

labour recruited from the local labour exchange. Except on an earlier job of much smaller magnitude, neither foreman nor labourers had had any experience of this class of construction. Work is normally carried out in one 8-hour shift, but when it is not possible to pour the quantity of concrete required by one lift overtime is worked as it was not considered desirable to introduce a second shift. Excavation for the first piers was commenced in February 1932, and the whole of the work was scheduled for completion last month.

Water Towers, Bunkers, Silos and Gantries.

THIS book,* the latest of the useful series on concrete and reinforced concrete published by Concrete Publications, Ltd., is a companion volume to "Reinforced Concrete Reservoirs and Tanks" by the same author, published in 1931. It is, however, self-contained and can be used without any reference to its predecessor so some repetition has been unavoidable.

The first part of the book deals in a thoroughly practical manner with the design and construction of the tank proper, the supporting structure being considered later. Several shapes are carefully treated, including rectangular, conical, and cylindrical tanks with flat, hemispherical, conical, and domed bottoms, and other more complicated shapes. Details of various methods of designing the bottoms, walls, and roof are given, with a consideration of their relative economy and with advice as to precautions to be taken to ensure watertightness. The effect of fixation of side walls by the bottom slab is treated in the way developed by Dr. Reissner.

A useful chapter is that dealing with the design of tanks for containing hot liquids. The author considers this problem in a sound manner, and points out that although the temperature gradient through a concrete wall may be found the stresses cannot be satisfactorily determined. The methods of stress determination proposed appear reasonable but, as the author points out, it is difficult to make a watertight concrete structure for large temperature ranges, especially where frequent alternations take place.

The second portion of the book concerns the design of bunkers, both shallow and deep. Such structures are very frequently met with in practice and the requirements and complications of size, shape, and variety of load call for the exercise of much ingenuity, and there are many ways of treating the design of such structures. In many textbooks on design methods are shown of arriving at the pressures and loads of the contained material on the side walls and bottom of the bunker, but the student or designer is left to find out for himself the best practical way of designing the structure to take these loads. The author goes farther, shows the principles to use in designing bunker structures of various shapes and follows them up by practical illustrations.

Following naturally after bunkers is a section in which the design of silos for corn, cement, etc., is dealt with. This section is perhaps too briefly considered, and could with advantage have been illustrated by more examples.

Then follows a chapter on the design and construction of the legs or supports which are required for water towers, bunkers, and silos. The treatment of such structures in a strictly mathematical way is a matter beyond the practising engineer, so approximate methods based on reasonable assumptions justified by successful use in the past are here put forward. A useful addition to this section would be the consideration of the stresses in the sub-structure of a tank supported on columns with a central shaft and stairway.

Before the final section on the design of gantries there are several chapters dealing with matters of particular interest and use for the young engineer or non-specialist designer. In one of these chapters the author explains the abbreviated methods adopted by some reinforced concrete specialists in preliminary design and estimation of quantities. In the next particulars are given of special types of doors and feeding apparatus for

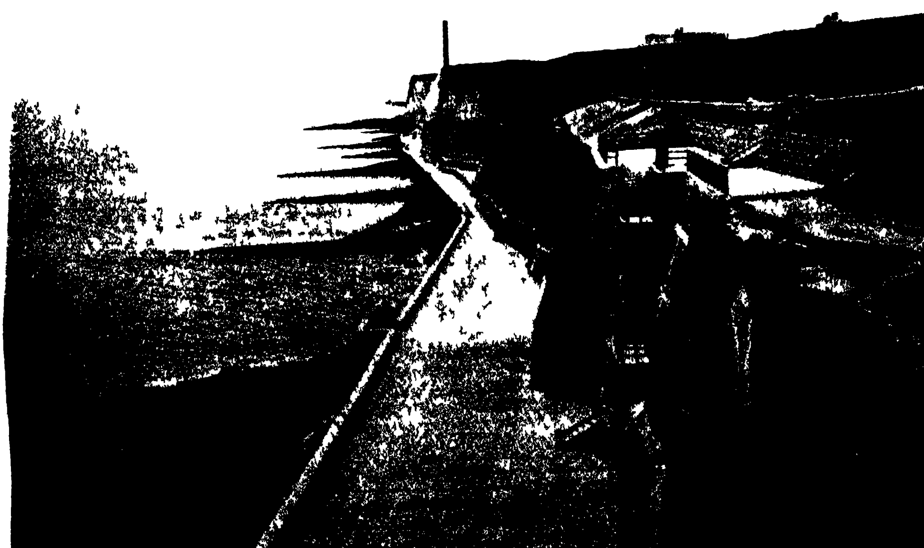
* "Reinforced Concrete Water Towers, Bunkers, Silos and Gantries." By W. S. Gray. 256 pp.; 170 illustrations. London: Concrete Publications, Ltd. Price 10s. (10s. 6d by post).

bunkers and silos. The author then devotes a chapter to the methods adopted by a contractor or specialist firm in the preparation of an estimate, the starting of the job and details of scaffolding and the placing of steel. These will be useful to the junior engineer, though their application is not restricted to the structures specified in the title. This chapter also contains a recommendation that bending lists (of which a typical page is shown with the drawing to which it refers) should be supplied to the contractor. Such bending lists are not always supplied by designers, and the work of taking off the steel is thrown upon the foreman or upon a member of the contractor's staff, who cannot be expected to do this work so well as the designer or his draughtsman. Most of the specialist designing and contracting firms have found such steel sheets or bending lists ensure quicker and more accurate placing of the steel and prevent misinterpretation of the drawings.

The section on the design of gantries is really too short to deal fully with a subject that one would rather expect in a book on bridges. This chapter contains some useful information which the author presumably felt might well be included, as it does not appear elsewhere in this well-known series of books.

The methods of design used in the book assume a knowledge of theory of structures and simple reinforced concrete, but abstruse mathematics has been avoided. Common sense and a true perception of practical limitations of the material have been used. Among the very few statements which will not meet with approval is the recommendation to use expansion joints every 300 ft. in gantries. So long a length of gantry exposed to the variations of temperature in this country would certainly crack. A more reasonable figure would be 100 to 150 ft. at most.

The book is one which should be available in every designer's office for reference by the seniors and for study by the juniors. A. I. B.



New Promenade, Rottingdean-Saltdean.

The promenade and sea defence works illustrated were carried out this year under the direction of Mr. D. Edwards, M.Inst.C.E., Borough Surveyor of Brighton. The promenade is surfaced with 6 in. of concrete reinforced with a single layer of B.R.C. fabric.

A New Form of Reinforcement for Concrete.

(*Contributed.*)

IT is well known that if ordinary grades of steel are worked cold their tensile strength is improved: in other words, the steel is "physically developed." Some sixteen years ago a paper was read in London before the Society of Engineers describing experiments and tests on mild steel bars physically developed by twisting them about their longitudinal axis, and it was clearly demonstrated that the tensile strength of the steel was materially increased.

The theory on which these experiments were based is sound, but when a single bar is twisted on its longitudinal axis no stresses are set up in the steel at the centre of the cross section of the bar (i.e. the axis of rotation), and hence there is no physical development at this point; also there is very little development in the area adjacent to the centre of the section, whilst at the outer edge of the section there is excessive development. Thus the bar is not uniformly developed throughout its cross section. In the early experiments square and triangular bars were twisted. The results showed that the triangular section was not so good as the square section which in turn was inferior to a round section, the reason being that the fibres in the outer parts of the cross section of the bar, being more remote from the centre and thus having to describe a larger circle than the fibres nearer the centre, became very brittle. In the case of the twisted round bar there is no visual indication of the amount of twisting the bar has undergone, hence the bars could easily be overdeveloped and rendered brittle; and also it would be difficult to distinguish such a bar from an untwisted bar.

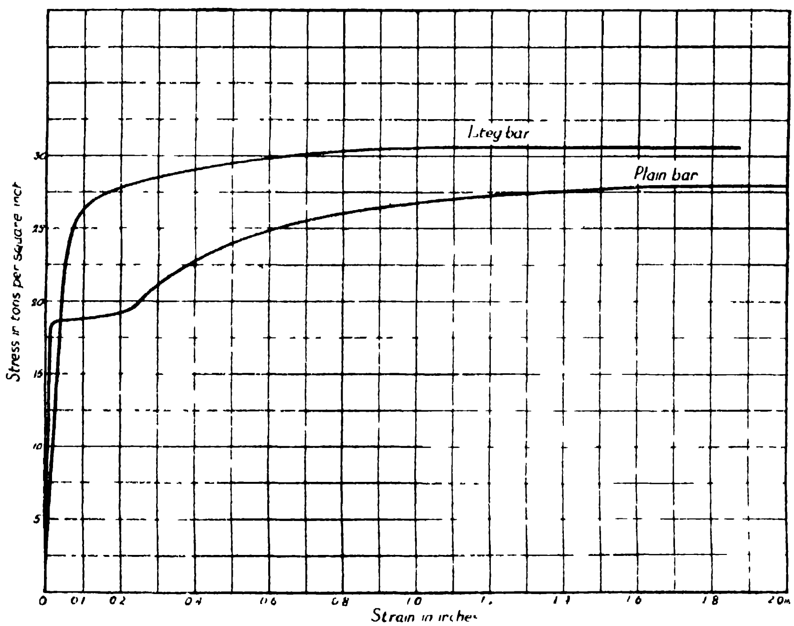


Fig. 1.

Yield Point.

If we draw a diagram of stresses and elongations for several mild steel bars of the same manufacture we find that the yield points vary within wide limits and

that, generally speaking, the diagram is more or less erratic. This is explained by the relative lack of uniformity in commercial quality steel bars.

The yield point being much lower than the ultimate strength of the steel, in a reinforced concrete structure the tensile breaking strength of the steel does not yield of a mild steel bar is considerably above this, consequently the ultimate breaking strength of the steel is never developed, since—long before it is approached—the relatively elastic steel has stretched more than concrete can stand.

Research and experiment in Central Europe have led to the development of

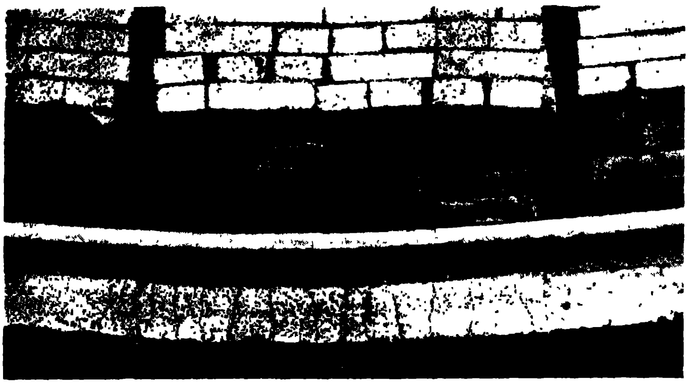


Fig. 2 (*a*).—**Failure of "Isteg" Beam.**

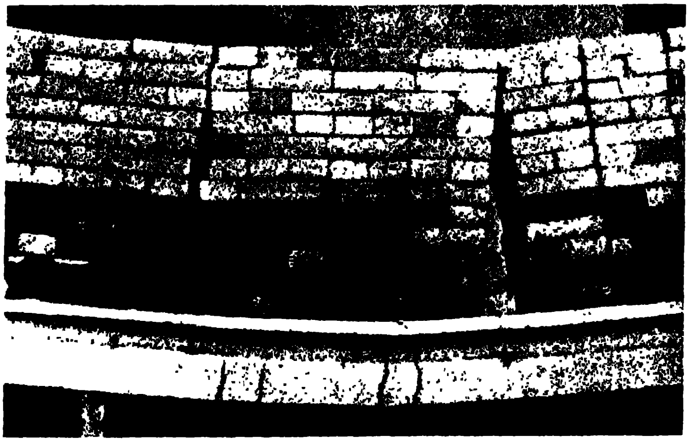


Fig. 2 (*b*).—**Failure of Beam with Ordinary Steel Reinforcement.**

come into question. Reinforced concrete is based on the co-operation of the two materials dependent on their proportional deformation. The steel can take up only a certain force resulting in an elongation with which the concrete can comply. It has been found that this elongation varies between 0·4 and 0·5 per cent. of the length of the steel rod. The normal the "Isteg" process, which by a simple cold mechanical process alters the elastic behaviour of the rolled bar and corrects deficiencies of manufacture and rolling. Its point of departure is the stretching of the steel before its use. It has been found that if two round bars of ordinary commercial mild steel are placed side by side and stretched, while being twisted

together, to a predetermined point beyond the yield point of the component rods, the resulting twin bar, whatever the yield points of the component bars may have been, will have a regular stress-strain diagram (see *Fig.* 1) giving a yield point (taken at 0·2 per cent. permanent elongation) of a minimum of 58,500 lb. per square inch (see *Table* II). With this twin bar (known as " Isteg " steel) the axis of rotation is the line at which the composing bars make contact throughout their length—hence the entire cross section of each bar is developed. The amount of elongation or stretching the section, have shown steel-breaking stresses exceeding 69,000 lb. per square inch.

With " Isteg " reinforcement the important factor, however, is increased safety. The steel is tested by the stretching and twisting process, each bar being submitted to at least twice the stress it would ever be called upon to stand in the concrete structure. The twisting has a levelling influence on the strength of the steel and softer parts of the bar are hardened, hence the regular stress-strain diagram Bars with inherent defects, such as piping, seams, laps, etc (*Fig.* 3), which

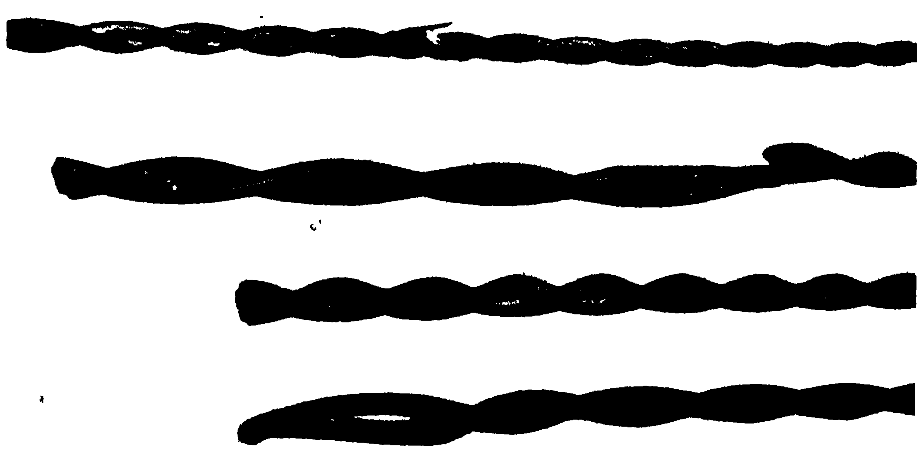


Fig. 3.

depends on the pitch of the spiral, which is regulated and can be checked at any time.

Assuming that an average concrete structure will withstand 0·4 per cent. steel elongation, over 80 per cent. of the breaking strength of the steel can be used. It is claimed that a structure reinforced with " Isteg " steel would not collapse owing to a sudden yielding of the steel, but due to a gradually increasing discrepancy between the deformation of concrete and reinforcement *Fig.* 2 (*a*) and (*b*). Numerous tests of reinforced concrete beams, so designed as to collapse due to excessive steel elongation, that is with a reinforcement of less than 1 per cent. of

have not been detected during inspection at the steel works, cannot withstand the twisting process and are rejected. Thus " Isteg " steel comprises separately tested reinforcing bars. The process guarantees a considerable increase in the stress at which co-operation between the concrete and the steel is maintained.

Although " Isteg " steel is new to England it has been used since 1928 on the Continent, where many thousand tons have been employed in concrete structures of all types, including the flooring for the tallest building in Vienna and a bridge in Czecho-Slovakia.

In August 1928 the City of Vienna, after conducting its own tests, permitted

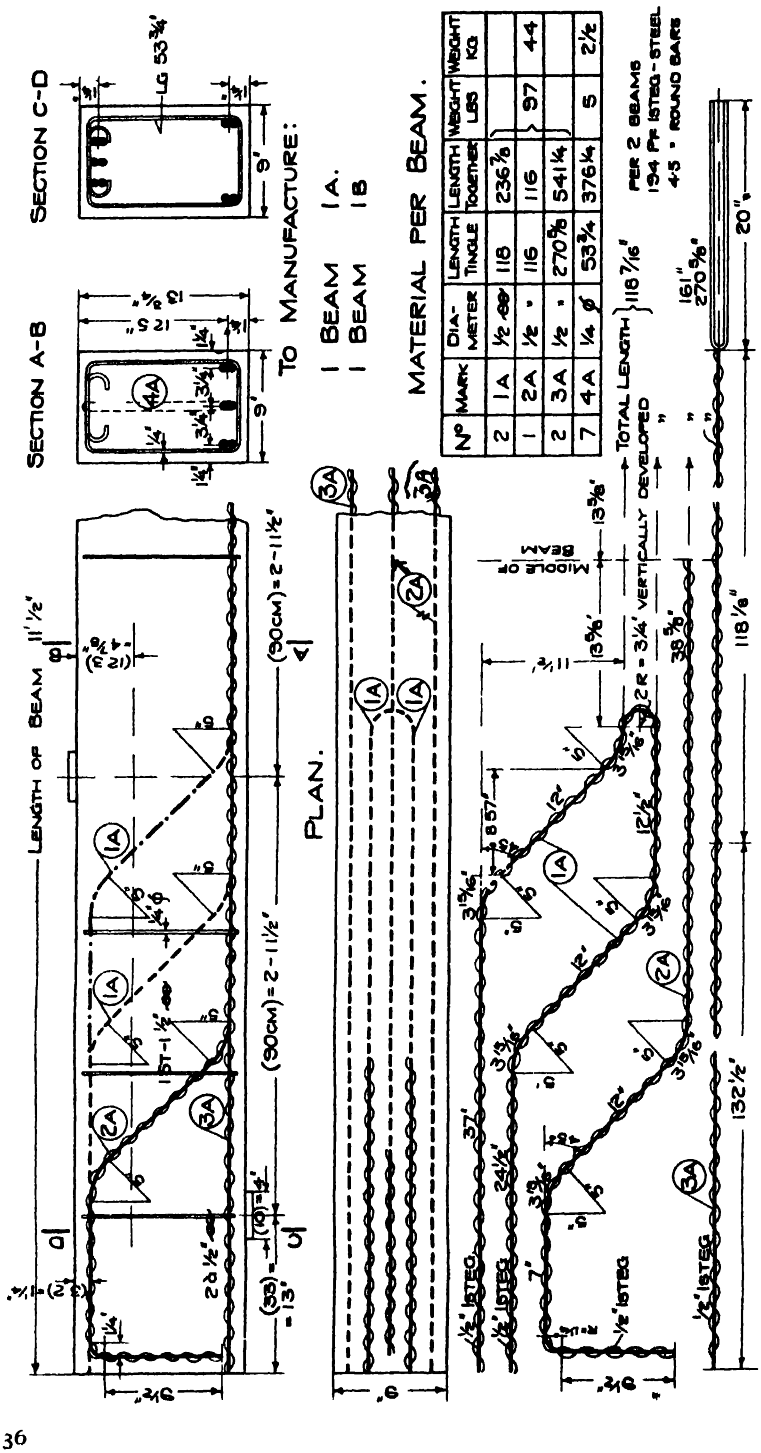

Section A-B
Section C-D
Length of Beam
Plan.
To Manufacture:
1 Beam 1A.
1 Beam 1B
Material per Beam.
Nº	Mark	Diameter	Length Single	Length Together	Weight Lbs	Weight Kg
2	1A	1/2	118	236 3/8		
1	2A	1/2 "	116	116	97	44
2	3A	1/2 "	270 5/8	541 1/4		
7	4A	1/4 ϕ	53 3/4	376 1/4	5	2 1/2
Total Length 118 7/16"
Vertically developed
Per 2 beams
194 Ft Isteg-Steel
45 " Round Bars
Middle of Beam
1/2" Isteg

Fig. 4.

"Isteg" steel to be stressed to 24,000 lb. per square inch. The Government of Czecho-Slovakia granted a preliminary decree for 24,000 lb. per square inch, and later, as a result of tests, the decree was amended in 1932 to permit "Isteg" steel to be stressed up to 25,500 lb. per square inch. The State of Prussia in February last, granted a decree for 25,500 lb per square inch as a result of tests conducted at Dahlem (Berlin) and at the University of Dresden These figures compare with 17,000 lb. per square inch allowed for ordinary mild steel bars in these three countries.

Tests with British Steel.

Tests were carried out in this country by Mr. R H. Harry Stanger in May last on four typical reinforced concrete beams, two of which were reinforced with "Isteg" bars and the others with ordinary bars In each case the steel was ordinary commercial quality supplied by Messrs. Dorman, Long & Co., Ltd., and both sets of bars were from the same cast Details of a test beam are shown in *Fig.* 4 In beams Nos 1 and 3 the reinforcement was ½-in "Isteg" steel (0·38 sq. in cross section), whereas beams Nos. 2 and 4 were reinforced with ⅞-in. round bars (0·60 sq. in. cross-sectional area). Each beam was 9 in. wide by 13¼ in deep and 11 ft. long, and composed of 1 : 2 : 4 concrete with a water-cement ratio of 0·92 by volume. The aggregates used were ¾-in Ham River shingle and Ham River sand Two 6-in. cubes crushed at 14 days gave a mean compressive strength of 2,009 lb. per square inch. In making the tests the beams were supported on bearers 3 in. wide on a span of 8 ft. 10½ in., and the load was applied at the third points of the span. The results of the tests are given in *Table* I. The beams reinforced with ordinary bars had 58 per cent. more reinforcement than the beams with "Isteg" bars, but the "Isteg" beam withstood the same load and gave a stress of 60,000 lb. per square inch for the steel

Tests carried out at the University of Prague 18 months ago, using similar beams with the same concrete section reinforced with "Isteg" bars of Czecho-Slovakian open-hearth steel, gave stresses of 68,000 lb. per square inch.

Tests made by Professor S. M Dixon on two beams 40 days old showed that a saving in concrete is produced by substituting "Isteg" reinforcement for ordinary mild steel bars In the tests the beams were 11 ft long, but the section of the beam reinforced with ½-in "Isteg" steel was 11½ in by 8 in while that reinforced with 1-in plain round bars was 12 in. by 9½ in The cross sectional areas of the concrete in the two beams were 92 and 114 sq in respectively, and their weights 1,100 lb and 1,350 lb. The maximum superimposed loads were 32,700 lb and 16,860 lb respectively, the "Isteg" beam, with 24 per cent less concrete in its cross section, supporting twice the load carried by that containing the plain reinforcement With the beam

TABLE I

Beam No	1	2	3	4
Reinforcement	½-in "Isteg"	⅞-in. round	½-in "Isteg"	⅞-in round
Age (days)	34	34	29	29
Weight (lb)	1459	1486	1489	1511
Total load at first microscopic crack in concrete on bottom of beam (tons)	5·5	8·5	7·0	10·0
Total load at which hair cracks were general along bottom of beam (tons)	18·0	14·5	18·0	14·0
Maximum total load (tons)	*21·5	22·5	22·4	22·39
Maximum bending moment (in. tons)	381·6	399·4	397·6	397·4
Maximum shear (tons)	10 75	11·25	11·2	11·2
Maximum tensile stress in steel, assuming modular ratio as 15 (tons per square inch)	29·7	21·3	30 9	21·2

*** During the test of beam No. 1 a load of 20 tons was maintained for half an hour. With beam No. 2 a load of 20 tons was maintained for ten minutes only.**

containing "Isteg" reinforcement hair cracks did not appear until the superimposed load reached 16,000 lb, whereas hair cracks occurred in the beam reinforced with plain steel at a superimposed load of 12,000 lb The corresponding deflections at these loads were 0 21 in and 0 14 in respectively At a load of 16,000 lb cracks developed in the 12-in by 9½ in beam, and in the 11½ in by 8 in beam, at a load of 24,000 lb, a noise was reported to have occurred inside the beam which was possibly due to failure of the reinforcement

Tensile Tests.

The stress-strain diagram (*Fig* 1) shows that for "Isteg" steel there is no abrupt yield point as is the case with ordinary mild steel It is therefore necessary to agree on a definition of the yield point The German Government in an official decree in February this year,

TABLE II

Results are given below of tensile tests (proposed by Dr Oscar Faber) with "Isteg" steel embedded in concrete (the cross section of the concrete not exceeding ten times that of the steel) Gauge length of tests was 24 in and the concrete section was 2 in by 2 in for ½ in "Isteg" bars and 3 in by 3 in for ¾ in "Isteg" bars Untwisted tag ends of the original steel were also tested these test pieces were turned down in lathe length of test piece 8 to 10 in

Test Piece				Yield Point						Ultimate Strength		
				Tag End taken at fall of the beam			Isteg taken at 0 per cent permanent elongation					
Tested Date	Size in	Remarks	Steel Mark	Tons per sq in	Lb per sq in	Percentage of original ultimate stress	Tons per sq in	Lb per sq in	Percentage of original ultimate stress	Tons per sq in	Lbs per sq in	Remarks
*1/6/33	¾	Tag end of original steel	5	[illegible]	40 750	75				[illegible]	[illegible] 500	Specimen turned
*13/6/33	¾	Tag end of original steel	5	[illegible]	41 [illegible]	[illegible]				8	63 [illegible]	(check test)
*8/6/33	¾	"Isteg"—embedded	5				[illegible]6 3	58 [illegible]00	91	30 3	[illegible]	
*13/6/33	½	Tag end of original steel	5	18 6	41 700	[illegible]8				[illegible]7 5	[illegible] 100	Specimen turned
*8/6/33	½	"Isteg"—embedded	5				[illegible]6 5	59 400	96½	30 1	67,400	
*8/6/33	½	Tag end of original steel	1	[illegible]0 5	45 900	67½				30 4	68 100	Specimen turned
*13/6/33	½	Tag end of original steel	1	[illegible]1 3	47 [illegible]00	7[illegible]½				[illegible]9 4	65 800	" " (check test)
*8/6/33	¾	"Isteg"—embedded	1				27 [illegible]	[illegible]1 800	9[illegible]½	31 8	71,200	
*8/6/33	¾	"	1				[illegible]8	63 100	91	31 6	71 [illegible]00	
*1/6/33	¾	Tag end of original steel	1	18 [illegible]5	40 [illegible]00	60				30 4	68 000	Specimen turned
*8/6/33	½	"Isteg"—embedded	1				[illegible]8 5	63 800	91	3[illegible] 4	7[illegible] [illegible]00	
*8/6/33	¾		1				[illegible]7 1	60 700	8[illegible]	32 1	71,800	
*8/6/33	½		1				27 0	60 500	91	30 4	68 100	Works test ultimate 66,600
*8/6/33	¾		1				[illegible]6 1	58,500	92½	[illegible]	[illegible]6 300	Works tests ultimate 63,200
*8/6/33	½	"	6				28 4	63 600	91	33 [illegible]	75 300	Works tests ultimate 69,900
*8/6/33	¾		6				28 1	62 900	91½	3[illegible] 8	73 400	Works tests ultimate 68 700
†21/6/33	¾	Tag end of original steel	3	0 26	15 380	61				31 87	71,390	Skin machined off test piece
†8/6/33	½	"Isteg"—embedded	3				30 0	67 200	91	35 7	77,800	
†8/6/33	¾		3				30 8	68 900		36 3	81 300	
‡22/4/33	½		5				27 7	62 000				
‡22/4/33	½		5				27 7	62,000				
‡22/4/33	½		1				26 5	59 300				
‡22/4/33	¾		1				28 1	63 000				

* Tested at City and Guilds Engineering College
† Tested by Mr R H Harry Stanger
‡ Tested by Federal Testing Station of Laboratory of Technology, Vienna

defined the yield point for " Isteg " steel as being the stress which produces a total elongation of 0·4 per cent. For the purpose of English tests the yield point has been taken at 0·2 per cent. permanent elongation.

If an " Isteg " bar is gripped in a testing machine and pulled in air a certain stress is expended in bringing the two strands of the " Isteg " bar into direct contact throughout the length of the test-piece. In the tests at Berlin and at Dresden University the German Government recognised this difficulty and agreed to take no measurement for elongation in the test-piece until the testing machine registered a stress of 7,200 lb. per square inch and this stress was called the zero point of the test. In England a test suggested by Dr Oscar Faber has been approved in which the " Isteg " bar is embedded in a small section of concrete, it being specified that the concrete section must not exceed ten times the section of the steel test-piece. Results are given in *Table* II of a series of these embedded tensile tests carried out by Professor S. M. Dixon at the City and Guilds Engineering College on " Isteg " bars made from steel supplied by five different British steel works; all " Isteg " bars were delivered for test with untwisted tag ends of the original steel. These tag ends were cut off and submitted to the usual tensile tests for yield point, ultimate strength, etc. The " Isteg " bars were then embedded in concrete and after three or four days were broken in a testing machine. It will be noted that the yield points for " Isteg " vary from 58,500 lb. up to 68,900 lb. as against yield points of the original steel (from which the " Isteg " was made) of from 41,700 lb. up to 47,700 lb. If the yield point of the " Isteg " bars is expressed as a percentage of the ultimate strength of the original bars it is found that in only one case did the " Isteg " yield point drop below 90 per cent, the average being well above that figure

Book Review.

" Mitteilungen über Versuche ausgeführt vom Österreichischen Eisenbeton-Ausschuss." Heft 13.

Vienna. Österreischer Ingenieur und Architekten-Verein. Price $1.60.

THIS is the thirteenth report of the Austrian Society of Civil Engineers and Architects dealing with the tests of a committee investigating the properties of reinforced concrete. The contents include a paper by Dr. R. Saliger on experiments on columns containing high-tensile steel reinforcement, and a report by Dr Emperger on the strengths of hooped columns Dr. W. J. Muller contributes an article dealing with the use of pipes made of special concretes, such as the Vianini and the asbestos-cement pipes The remainder of the volume is composed of a paper on the standardisation of aggregates and an article describing tests of concrete slabs reinforced with expanded metal.

Construction in Winter in Canada.

AT the University of Manitoba a new science building of reinforced concrete has been completed without the usual stoppage of construction during the winter.

The building is 230 ft. long with end wings extending backward about 100 ft. It contains four stories and a basement, with a flat roof concealed by a parapet. The building is of reinforced concrete construction throughout, in large part of beam-and-slab type, but in certain parts of slab-and-joist type to make the ceilings flat; the structure rests on a piled foundation. The walls are faced with quarry-faced limestone, laid irregularly and with a pleasing variety of size, surface, texture, colour and jointing.

According to " Engineering and Contract Record " the construction of the building progressed continuously during the winter within an enclosure of ½-in. insulation board. Within this box, which was warmed to about 45 deg. by air blown over steam coils, operations, including the stone masonry, were carried on without regard to outside temperature or weather conditions

Deterioration of Structures in Sea Water.

In the latest report * of the committee of the Institution of Civil Engineers which has been investigating the action of sea water on structures of timber, metal, and concrete details are given of the final examinations by Dr J. Newton Friend of the second series of iron and steel specimens which have been exposed at Halifax, Nova Scotia, and Auckland, New Zealand, for ten years The results so far obtained show that the cast irons, while apparently sound, may have been seriously weakened by internal corrosion, and that the 13·57 per cent chromium steel appears to be quite unsuitable for conditions involving immersion in the sea owing to the severe localised pitting to which it is subject Of the wrought irons, the Low Moor iron is in general the best in all positions of exposure The mild steel with low manganese and high sulphur and phosphorus was in some cases in good condition and in others very severely attacked. It thus appears to be an unreliable material. The tests show that the advantages of adding small proportions of copper or nickel to mild steel are less marked after ten years' exposure than after five years The removal of mill scale before the metal is exposed to the sea is also useful The wrought irons were cleaned before exposure and should therefore be compared with the cleaned mild steels when the respective merits of the two metals are judged. At both Auckland and Halifax the aerial mild steels in general lost less in weight than the aerial wrought irons, but, in the case of the other two sets, at Auckland the wrought irons as a whole were less corroded than the mild steels, while the reverse was observed at Halifax where corrosion had proceeded to a more advanced stage

The remaining set of painted steel plates, which had been placed in the open air at Southampton docks in 1924 has now been cleaned and examined Coatings that proved to be satisfactory include oil paints, particularly the red leads, refined coal-tar, and zinc

Periodical examinations were made in March and September 1931 of the reinforced concrete test-piles exposed in duplicate, one set in sea water at Sheerness and the other in artificial sea water at the Building Research Station, Watford At the time of the latest examination the piles had been exposed for periods ranging from 24 to 29 months. Within the last year appreciable deterioration has become evident in a considerable number of specimens, cracking having occurred in several piles having 1 in. of cover over the reinforcement. With the exception of Portland cement-trass mixtures at Watford, all the rich mixtures are free from cracks The Portland blast-furnace cement mixtures at Sheerness are sound, though cracks have appeared in three of the piles made of lean mixtures of the same cement and exposed at Watford The disintegration of the lean aluminous cement mixtures is continuing

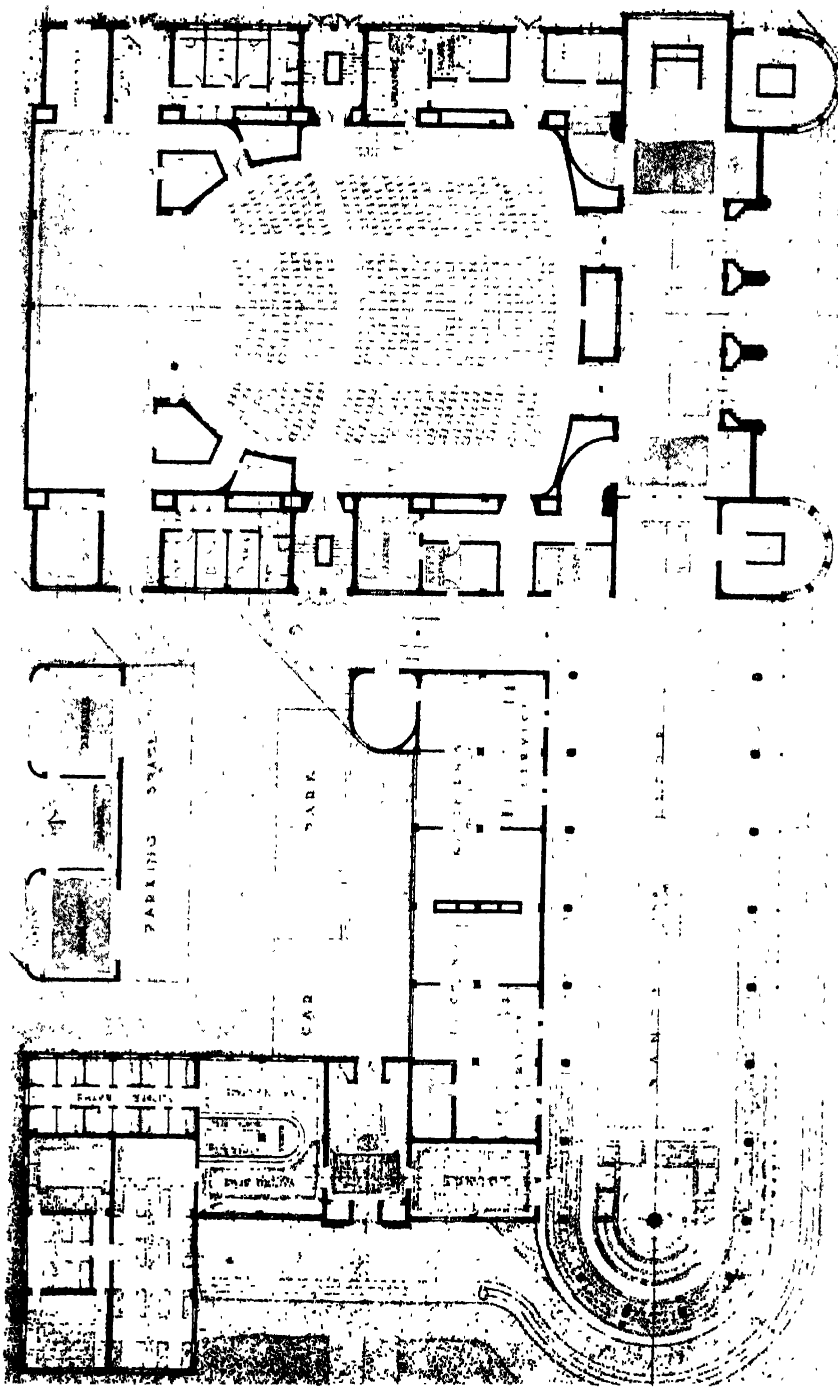
The reinforced-concrete piles exposed to sea action at Sekondi (Gold Coast), which were made of a selection of the mixtures employed for the piles exposed in England and in which the cover over the reinforcement is only 1 in., were examined after having been in position for 24 months A few rust spots and slight spalling are evident on a number of the piles, while the specimens made of aluminous cement mixtures are somewhat severely cracked at their upper ends.

The successful use of aluminous cement is reported by Mr. W. T Halcrow, M Inst C E, in a reinforced concrete piled wharf at Kinlochleven. The concrete, which was of 5 : 1 mixture, was placed in 1928, and, though the minimum depth of cover is no more than 1½ in, the work showed no sign of deterioration in July 1931.

Mr. S. H. Ellis, M.Inst.C.E., has furnished particulars of a number of reinforced concrete test-piles that have been partly immersed in sea water at Hong Kong for 11 years, together with information regarding the deterioration of reinforced concrete structures near the mouth of the Prai river. Mr. Ellis's observations point to the conclusion that, in order to obtain adequate protection of the steel reinforcement in a damp tropical climate with good quality 1 : 1½ : 3 concrete, at least 2 in. of cover is necessary. When a richer concrete was employed satisfactory protection was afforded by 1½ in. of cover.

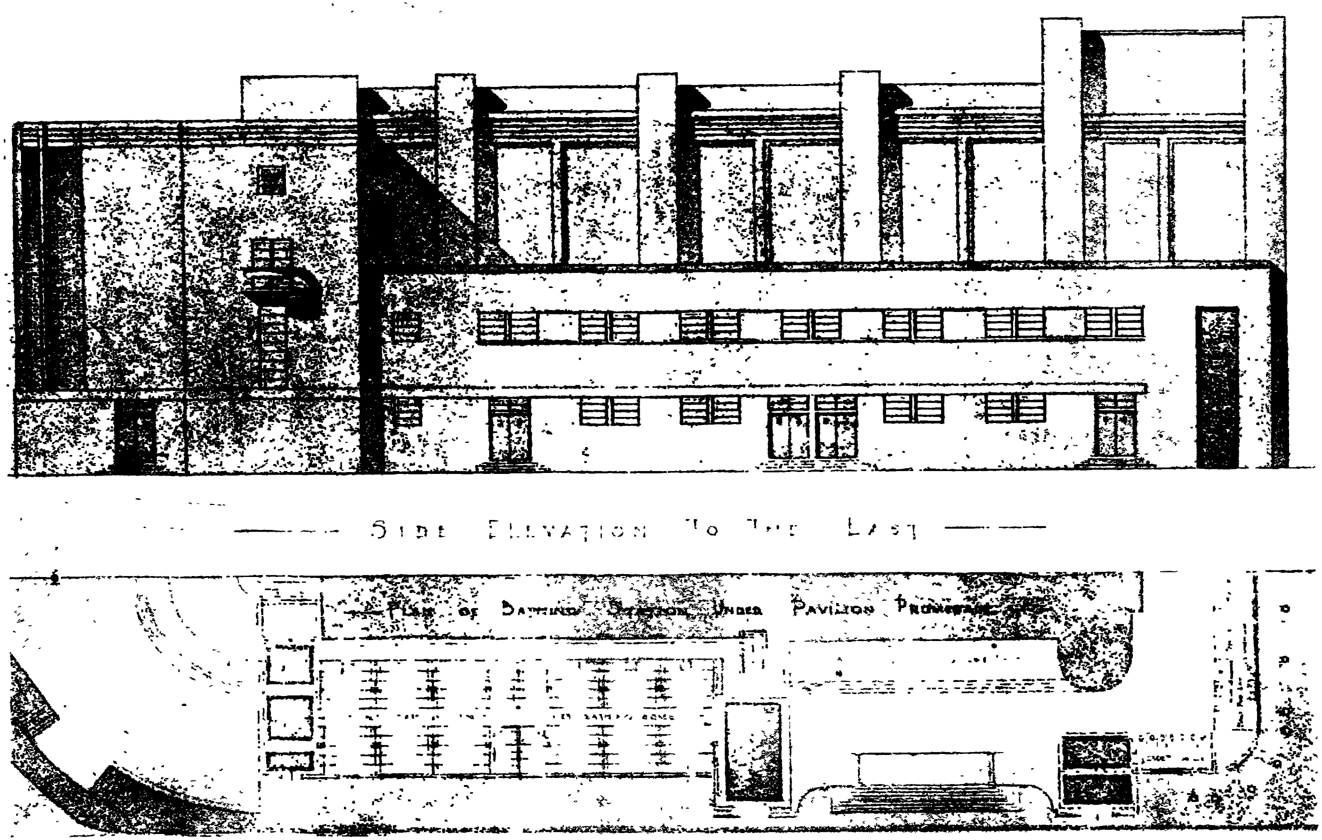
* "Deterioration of Structures in Sea-Water" London. H.M. Stationery Office Price 1s. 6d net

"Concrete and Constructional Engineering" Prize Design.



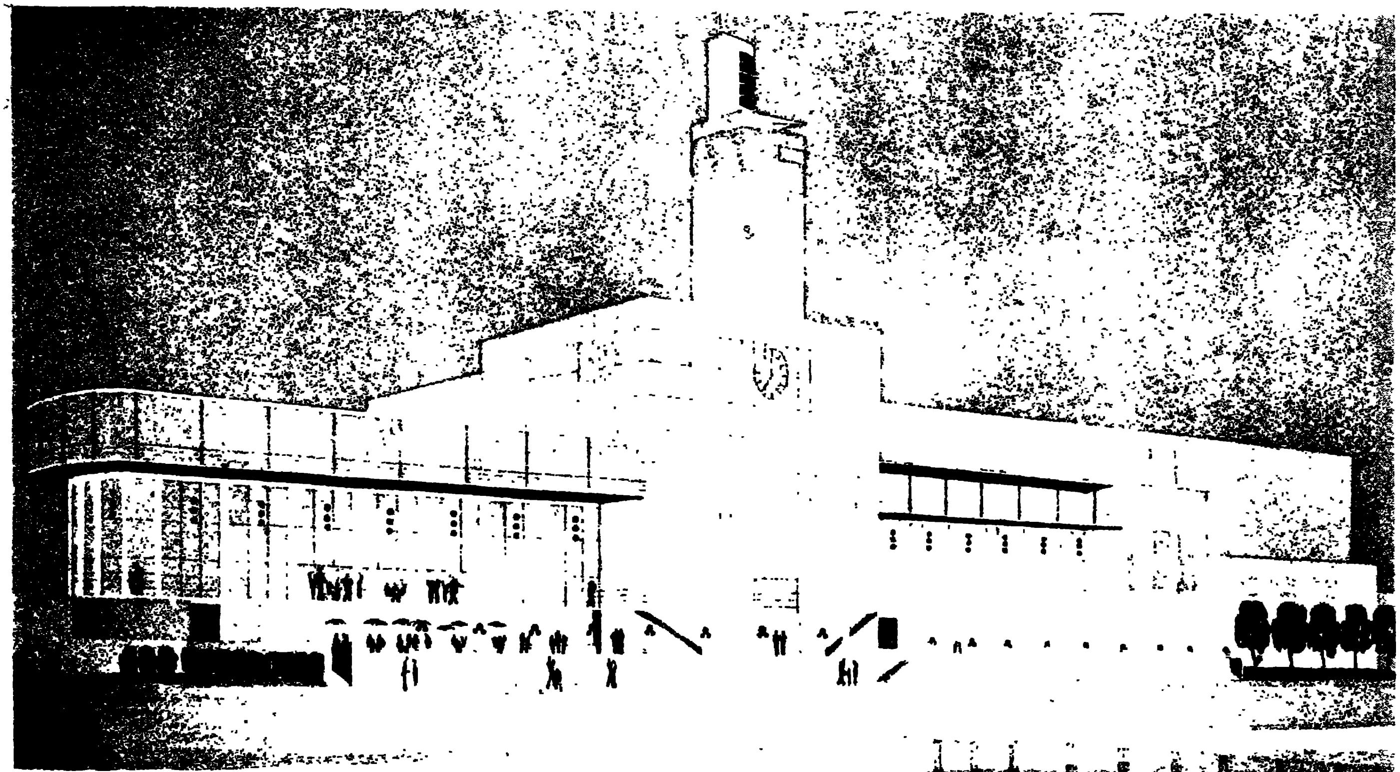
Design for a Seaside Pavilion, Ground Floor Plan. By Mr. E. C. O'Farrell. Awarded First Prize. (*See pp.* 508, *and* 542, 544-5.)

" Concrete and Constructional Engineering " Prize Design.



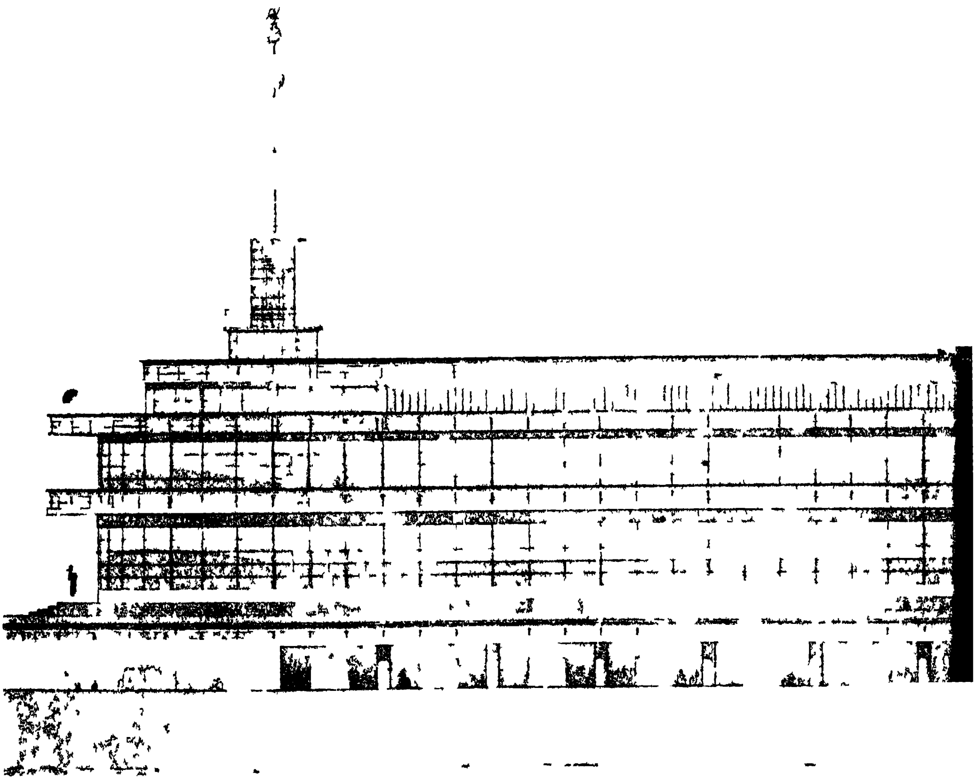
Design for a Seaside Pavilion. By Mr. E. C. O'Farrell. Awarded First Prize. (*See pp.* 541 *and* 544–5.)

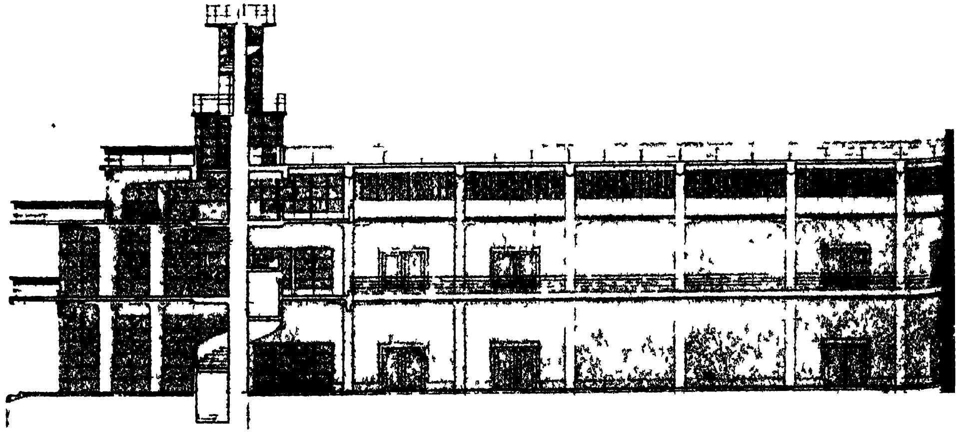
"Concrete and Constructional Engineering" Prize Design.



Design for a Seaside Pavilion. By Mr. A. P. Ciregna. (Second Prize.) (*See pp.* 508 *and* 546.)

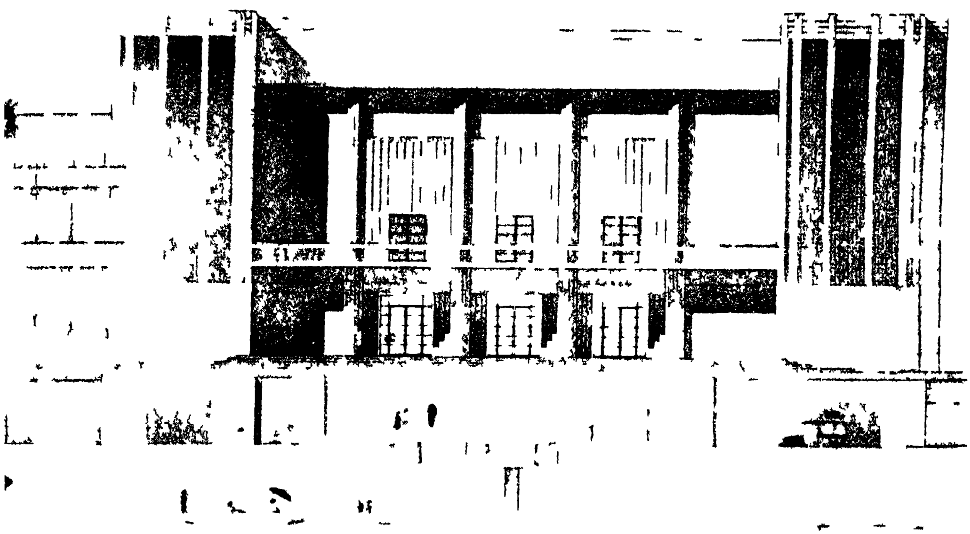
" Concrete and Construction





Design for a Seaside Pavilion. By Mr. E. C

Engineering " Prize Design.



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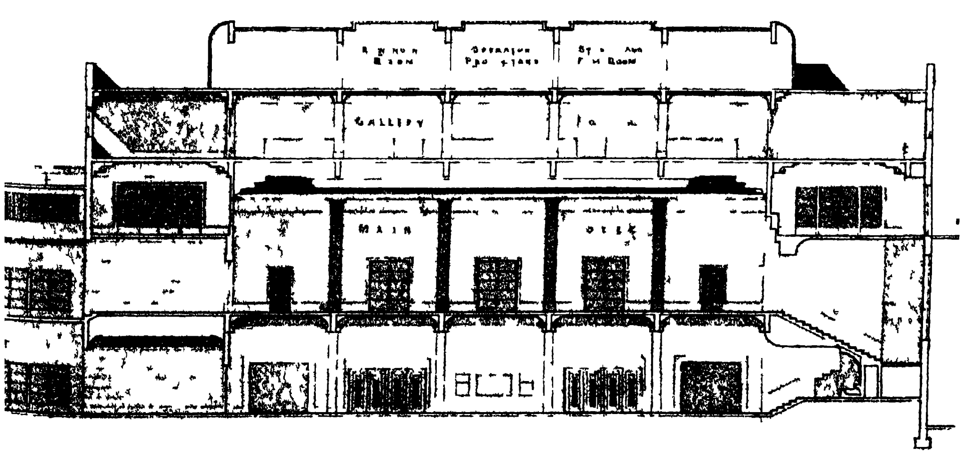
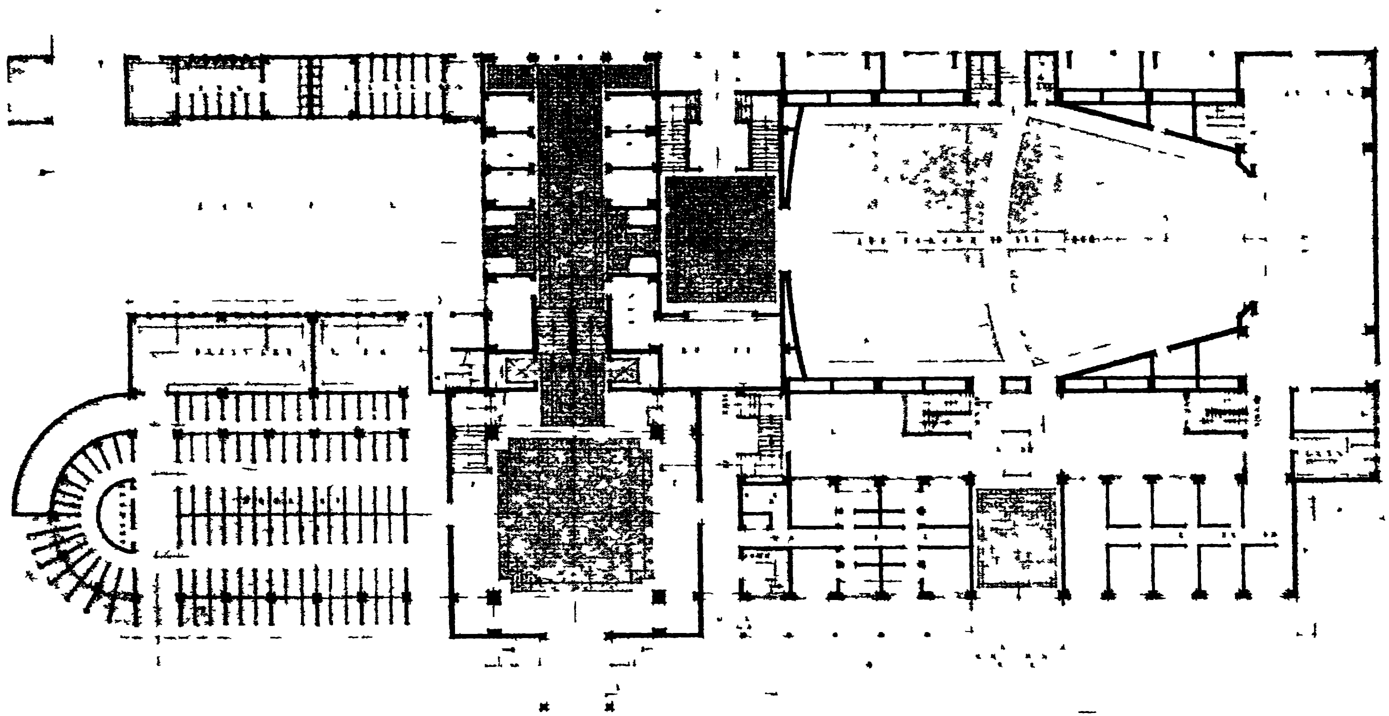


(TION

(Awarded First Prize.) (*See also* pp 541-2)

"Concrete and Constructional Engineering" Prize Design.



Design for a Seaside Pavilion, Ground Floor Plan By Mr A P Ciregna. (Second Prize.) (*See also p* 543)

The Storstrøm Bridge.

A CONTRACT was signed on May 14 by Messrs Dorman, Long & Co, Ltd, and the Danish State Railways for the construction of a combined rail and road bridge, about two miles in length, across the Storstrøm, a smaller bridge over the Masnedsund which is about one eighth of a mile wide, and about six miles of railway and roads, including the bridge approaches

These bridges and their approaches will carry the projected railway and road across the two straits, the Storstrøm and Masnedsund which separate the islands of Falster, Masnedo, and Zealand thus forming a link in what will be a chain of rail and road communications between Copenhagen on the north-east coast of Zealand with the southernmost point of Falster, leaving only the 25 mile ferry journey between that point and the mainland of Germany to connect Copenhagen with Germany, thus saving about an hour on the journey between capitals The bridges will carry a roadway approximately 18 ft 6 in wide between kerbs, a footway 8 ft wide, and a single-track railway of 4 ft 8½ in gauge

The design for the Storstrøm bridge (*Fig* 1) contains fifty spans, rising from each end towards the centre at a gradient of 1 in 150, and providing a clear head room for shipping of about 85 ft at the three navigation spans The centre navigation span is about 450 ft between piers and the two side spans are about 340 ft The remaining forty seven approach spans are alternately 190 ft and 204 ft between piers The navigation spans are stiffened tied arches, the arch ribs being of box section type, about 3 ft deep, while the stiffening girders are single web plate girders 12 ft deep over the flange angles The distance between the girders is about 30 ft The hangers are of built up joist section spaced at intervals of 30 ft to 35 ft Plate cross girders 4 ft deep are framed into the stiffening girders at the hanger points The railway and roadway decks are carried by six lines of joist stringers framed into the cross girders, and the footway is carried on two lines of joist stringers supported by brackets cantilevered from the cross girders

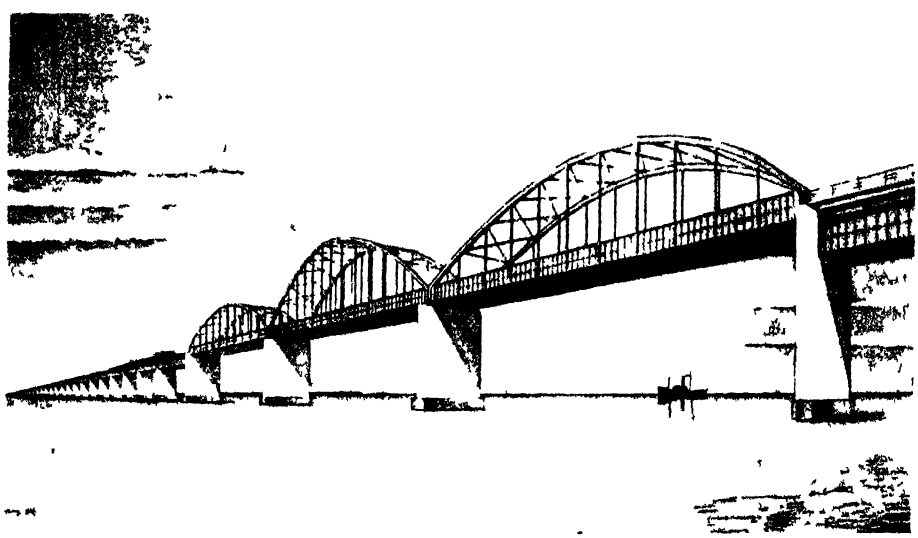


Fig. 1.

The roadway and footway are of reinforced concrete slab construction, the roadway being surfaced with 3 in of asphalt and the footway with 1¼ in of asphalt The railway track is carried on

timber transoms with planking to form footways on each side of the track. Lateral wind-bracing systems are provided in the plane of the arch ribs and at the level of the bottom flange of the stiffening girders The approach spans are of the cantilever type, with suspended spans and anchor arms located in alternate openings, the main girders being plate girders 12 ft. deep over the flange angles and 24 ft. apart. There are two hinges, situated 30 ft from the supports, in each of the longer spans, so that the whole construction consists of a series of girders with double cantilevers and 150-ft suspended spans between It is therefore statically determinate The cross girders are 3 ft deep and spaced at 14-ft 6-in centres, and bear directly on the top flanges of the main girders Two joist stringers carrying the railway are framed into the cross girders, and four roadway joist stringers rest directly on the top flange The roadway is of the same construction as on the navigation spans, but the railway track is laid on ballast carried in a reinforced concrete trough and the footway is cantilevered in reinforced concrete from the roadway slab Lateral wind-bracing is provided in the planes of the top and bottom flanges of the main girders. This bridge when complete will be the longest in Europe

The Masnedsund bridge design comprises six spans of about 100 ft each, one of which is an opening span of the fixed trunnion-bascule type The two main girders of each span are plate girders, 8 ft. 4 in deep, spaced about 38 ft apart The arrangement of cross girders and stringers and the bridge floor are similar to that on the navigation spans of the Storstrom bridge

Of the 30,000 tons of steel to be used the larger portion will be Messrs Dorman, Long's new high-tensile steel, Chromador steel, the adoption of which has considerably lowered the cost of the bridge. It will be manufactured in the company's works at Middlesbrough.

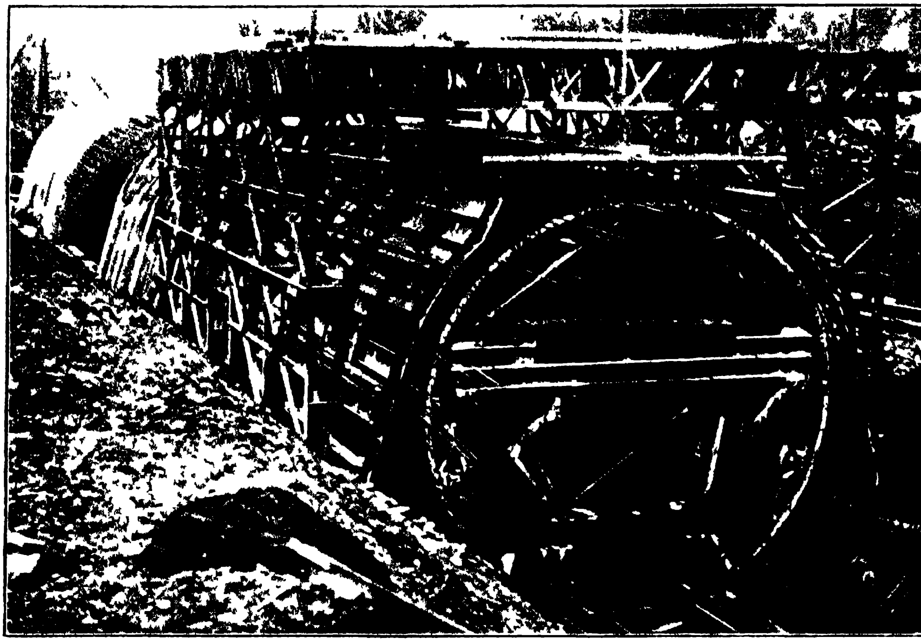
The works will contain about 160,000 cb. yd. of concrete and about 2,600,000 cb. yd. of earthworks Although Messrs. Dorman, Long & Co. are responsible for the whole contract, they will manufacture and erect the steel work only. The remaining portions of the various works will be undertaken by Messrs. Christiani & Nielsen, the agents in Denmark for Messrs. Dorman, Long & Co., and sub-contractors for the sub-structure and all other concrete construction in the Storstrøm and Masnedsund bridges and all works in connection with the bridges. The contract specifies that the whole of the work shall be completed in 4½ years, and the total cost is estimated to be £2,000,000

The Sub-structure.

The superstructure is supported on fifty-one piers, all of concrete or reinforced concrete, and the maximum depth of the Storstrom on the line of the bridge is 46 ft The average depth is approximately 22 ft, and only for a length of approximately 2,000 ft. is the depth more than 30 ft.

On the shore on each side are several layers of mud which have no supporting value, but at a depth of between 20 and 25 ft. there is a layer of clay with a considerable amount of boulders Limestone rock is found at a depth of 65 ft to 130 ft, but the foundations are not carried down to this depth, as from the borings previously made by the Danish State Railways it is estimated that the supporting power of the clay is sufficient It is calculated that the greatest depth of the foundation will be 55 ft, while the average depth is approximately 35 ft. to 40 ft. The permissible pressure on the ground has been fixed at 3·2 tons per square foot. Before constructing the piers a number of borings will be made to ascertain the exact depth required for the foundations.

Conditions for the erection of the bridge are comparatively good. Tides are, on an average, not more than approximately 8 in to 13 in., and it is seldom that high water exceeds the level + 3·00 The velocity of the stream is sometimes considerable, but seldom more than 7 ft. per second, and since the sound is sheltered large waves will hardly ever be encountered, but west or south-west gales may cause rough seas. On the other hand conditions with regard to floating pack ice are difficult, as in severe winters floating packs of very great size may form and cause difficulties whilst the piers are under construction. It is therefore important that a method should be used in the construction of the bridge whereby a pier when once started may be speedily completed.



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Pier Construction.

The method of construction of the piers is as follows: Both on Masnedø and Falster the abutments adjoin very high embankments, the tops of which are approximately 60 ft. above water level. The abutments are of reinforced concrete and consist of a system of hollow cells resting on a thick mass-concrete raft. The external, as well as the two secondary longitudinal walls and cross walls, support the bridge superstructure which is cantilevered back so that the distribution of the load on the foundations is approximately uniform. These abutments will be made within a cofferdam of steel sheet piling which will remain in the building as a protection against undercurrents

The forty-nine intermediate piers are to be made of mass concrete, the foundations consisting, on the average, of a 10-ft. thick concrete slab, elliptical on plan At the water line, from the level − 8·00 to + 8·00 the piers will be covered with granite as a protection against water and floating ice. Above level + 8·00 the piers will be of cellular construction. Around all the foundations steel sheet piling is to be driven to 10 ft. to 15 ft. below the foundation raft, and will act as a safeguard against undermining of the piers by the current. The possibility of further protection by stone filling has also been taken into consideration

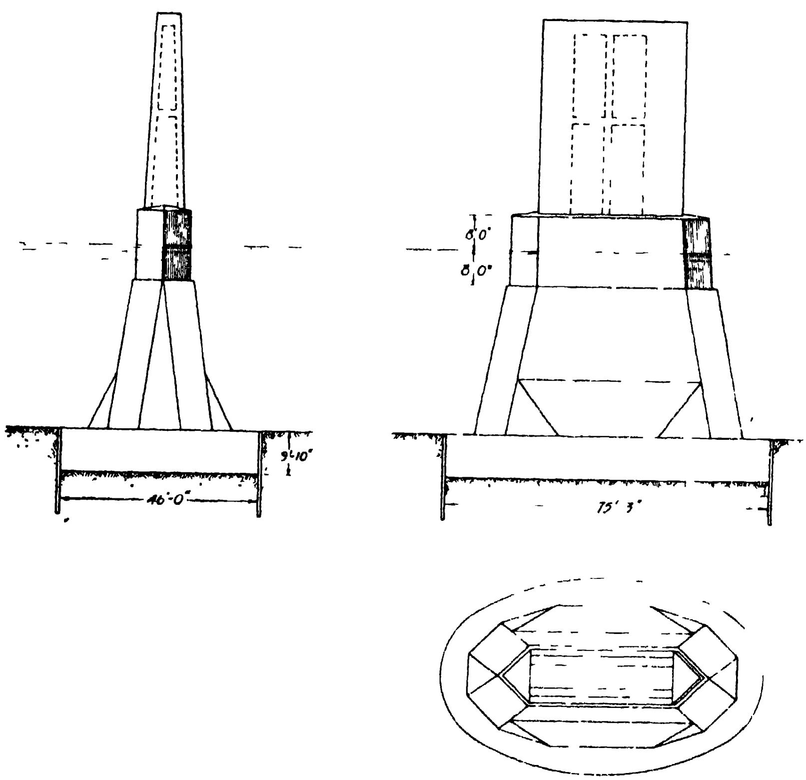

Fig. 2.

The great number of intermediate piers standing in nearly the same depth of water and subject to practically the same forces makes it possible to standardise the execution of this part of the work, and the piers are divided into a few standard types. The part of the pier which is under the water can be made the same for all piers of the same type, and the same shuttering can be used for all. The four piers at the navigation openings in the centre of the bridge, where the con-

struction of the superstructure is different, are necessarily of considerably greater dimensions than the others. Some piers standing in shallow water are smaller and not suitable to be made as standard types. Those at the centre openings and in the shallow water will be constructed in an open excavation inside a cofferdam of steel sheet piling. When the piers are built up to a level just above the water, the steel sheet piles will be cut off under water at the level of the top of the foundation raft.

The remaining piers, approximately forty in number, will be built with the aid of special standard units. Each of these is a floating body built of steel, the base being elliptical on plan externally, corresponding in size to the elliptical pier base; internally these structures will be made to act as shuttering for the lower part of the pier shaft. These units will first be used as staging for the driving of the steel sheet walls round the foundation. The steel sheet walls are only to be driven down to the length in which they will be required in the finished construction, and the units are sufficiently strong to serve as strutting for the steel sheet walls and as cofferdams for the pier shaft when the water in the units is pumped out after the sheet piles have been driven and tightened against the units.

The units also support the machinery necessary for keeping the pit dry, for excavating for foundation, etc., and the shuttering for the top of the piers, whilst they act as centering for the lower part of the piers. The units are also fitted with water tanks by means of which they can be raised or lowered by pumping water out or in.

When a pier has been constructed to a certain height above the water level the unit will be removed and the top part of the pier will be constructed in the ordinary way. As an example of the construction of a pier which will be built by means of the unit described, the illustration (*Fig.* 2) gives the main dimensions of the largest standard type.

For the sub-structure of the Storstrøm bridge 12,000 cb. yd. of concrete will be required, whilst for the bridge slab, which is in reinforced concrete, 12,000 cb. yd. will be used. It will be necessary to erect large floating plants to place this quantity of concrete.

The comparatively short time of 4½ years for the building of all the bridge work, including the time required for the more detailed examination of the stream bed, will make it necessary when the work is in progress to finish the lower part of a pier in approximately seven weeks.

Recent Patent Applications.

393,429.—Holzmann Akt.-Ges. Zweigniederlassung Bclin: Manufacture of light concrete.

393,576.—E. Freyssinet: Process and apparatus for manufacturing reinforced concrete sections.

393,605.—G. Jaeger, and J. Eggert: Device for supplying water to concrete mixers.

393,641.—Garvenswerke Maschinenpumpen und Waagen-Fabrik W. Garvens: Apparatus for the production of concrete piles.

393,864.—C. F. Ball: Concrete mixing and agitating apparatus.

394,505.—L. M. Fairclough: Concrete flooring.

395,256.—H. Camus: Shuttering for concrete.

395,368.—G. Carpenter: Construction of walls, ceilings and partitions.

395,442.—A. Grant: Hinge for obtaining flexibility of formwork for concrete construction.

New Companies Registered.

Oak Farm Sand and Ballast Co., Ltd. (278591.) Registered August 8, 1933. Oak Farm, Worton Road, Isleworth, Mdx. Nominal capital: £100.

Bradmere Hill Sandpit, Ltd. (278629.) Registered August 10, 1933. Burnham Norton. Nominal capital: £100.

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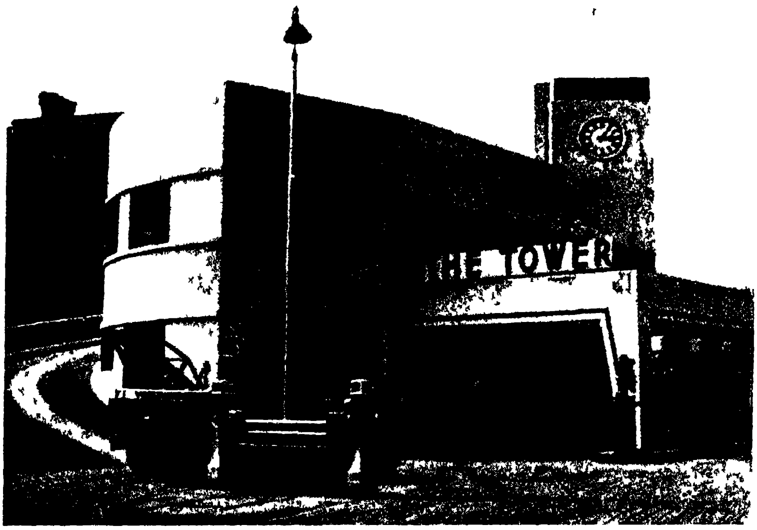

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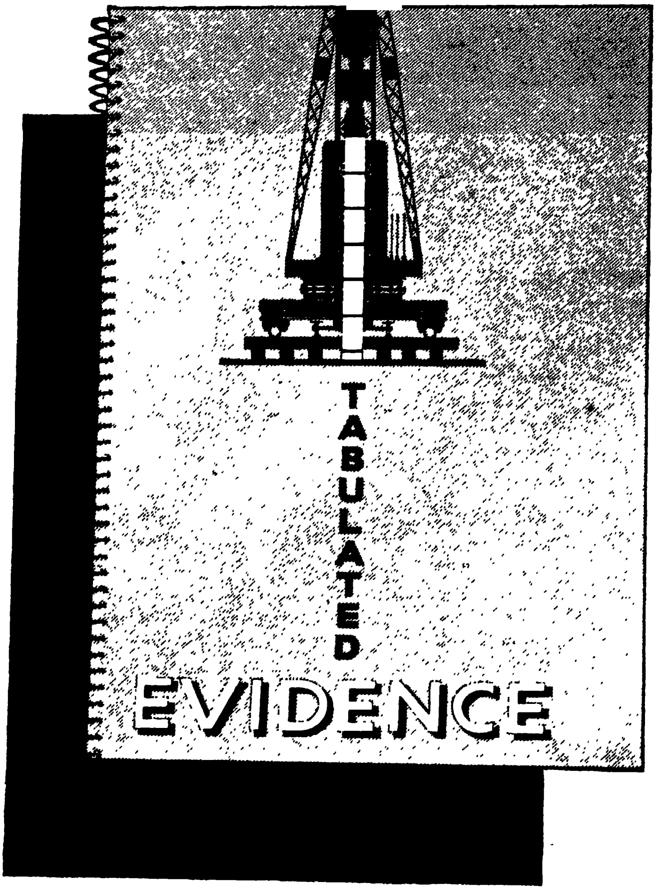


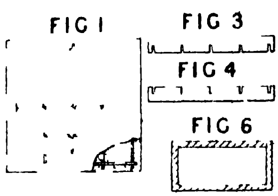
This new booklet gives tabulated data of many interesting contracts carried out by West's Rotinoff Piling & Construction Co., Ltd. (Regent House, Kingsway, W.C.2); a copy will be sent on request to anyone interested in piling.

Recent Patents Relating to Concrete.

Reinforced Blocks, Bricks, etc.

369,241.—Berry A F Wellington House Strand London Oct 10 1930

Blocks, slabs, bricks etc of concrete rendered cellular by internal generation of gas are strengthened by reinforcements which extend through or around substantially their whole length breadth and thickness The reinforcement may be internal for example interlocking metal strips as in *Figs* 1 3 4 or external for example a baked clay container with a lid (*Fig* 6) or both Other materials such as asbestos concrete may be used for the reinforcement which may take other forms e g hollow wavy angular The concrete consists of Portland or other cement mixed with broken burnt clay or asbestos etc and a small quantity of finely divided alloy or metal e g aluminium with optionally a chemical agent such as sodium hyposulphite The concrete is wetted packed into or around



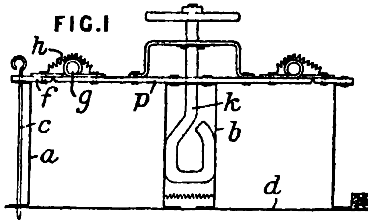
REINFORCED BLOCKS BRICKS ETC

the reinforcement and allowed to rest preferably in a warm state while gas is generated The unset blocks may be enclosed in a chamber and subjected to (1) reduced pressure to promote coarseness of cell (2) warm air for drying (3) a fluid hardening medium e g a solution of magnesium silico fluoride or sodium silicate The blocks may be covered with cement plaster paint etc veneer leather or fabric

Casting Walls in Situ.

369,285.—Schmidt J T Nebel on Amrum Germany June 8, 1931

Concrete walls are cast course upon course between movable metal moulds which are raised immediately after the casting of one course and are supported on that course The courses are formed of layers not more than 25 cms in height and the aggregate used is only slightly moistened The outer moulds comprise flanged metal plates (*a*) supported on cross bars (*d*) on the previously cast course and upper distancing members (*p*) are used on the moulds pins (*c*) passing



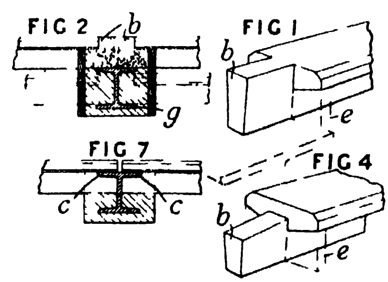
CASTING WALLS IN SITU

through the ends of these distancing members and through apertures in the flanges of the moulds The end parts (*f*) of the upper distancing members are secured to the moulds and are hinged to the main part (*p*) and springs (*h*) are used to pull these parts upward and so release the moulds from the cast surfaces as soon as the pins (*c*) are withdrawn For hollow walls core plates (*b*) are fixed to the upper distancing members and these are expanded by a rotatable key (*k*) and are spring retracted

Pre-cast Beams for Floors, etc.

370,164 Morton B and Morton H D Edgeworth Hawley Lane Hale Cheshire March 17 1931

Floor or roof structures composed of pre cast concrete beams of T section in which the upper webs form the entire



PRE CAST BEAMS FOR FLOORS ETC

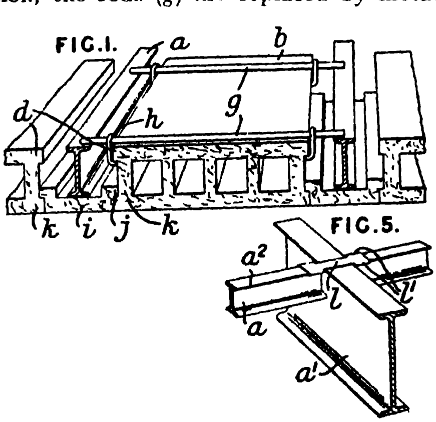
floor surface have at each end a web like extension formed by omitting parts of the top flanges or by slotting the beam from each end, the beams resting by

their extensions on supporting girders or their haunching and being fixed in position by grouting filled into the spaces between the extensions. *Fig.* 1 shows a form in which the extension (*b*) is formed of the whole depth of the web. In *Fig.* 4 the extension (*b*) is of less depth and has a sloping upper surface; the upper surface may also be horizontal. *Fig.* 2 shows the beams resting on a girder (*g*) and fixed by grouting. The beams may be formed with lateral flanges (*c*, *Figs.* 2 and 4) which will avoid the need for shuttering for retaining the grouting, or pre cast blocks cemented in position may be used instead of grouting. *Fig.* 7 shows a form in which the web is slotted at (*c*), the beam being supported on haunching. The upper surface may be flat or slightly arched, and the edges may be furnished with inter-engaging tongues and grooves. The undersides may be covered by expanded metal, etc., to form a ceiling.

Floors.

371,978.—W. G. Shipwright, 102, Abbey House, Victoria Street, London Feb. 11, 1931.

Hollow tiles (*b*, *Fig.* 1) are supported from joists (*a*) by metal rods (*g*) having members (*h*) which engage under the top flanges (*d*) of the tiles (*b*). In a modification, the rods (*g*) are replaced by metal



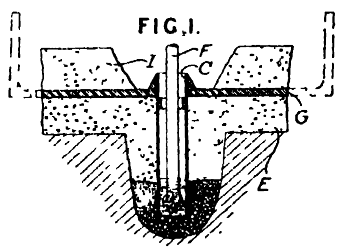
FLOORS.

plates having tongues which engage under the flanges (*d*) of the tiles (*b*). Further tiles (*i*) may be employed to fill the spaces between the tiles (*b*), these tiles (*i*) being formed with flanges (*j*), which rest on bottom flanges (*k*) of the tiles (*b*). The joists (*a*) are supported from main beams (a^1) by metal or other plates (*l*, *Fig.* 5), which rest upon the main beams (a^1) and have their ends (l^1) bent to engage under the top flanges (a^2) of the joists (*a*), or the plates (*l*) may be replaced by rods formed with hooks which engage under the top flanges (*d*) of the joists (*a*).

Waterproof Floors.

372,709.—T. G. Marriott, Artillery House, Artillery Row, Victoria Street, London. Jan. 9, 1932.

Waterproof floors and linings for chambers below water level are formed by laying first a concrete or like bottom layer (*E*) with pipes (*C*) extending through into the subsoil, so that water may be removed during construction by means of a hose (*F*)



WATERPROOF FLOORS.

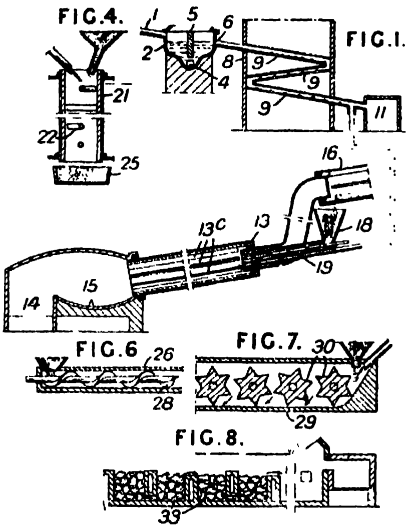
inserted into the pipes An asphalt or similar dampcourse (*G*) is then laid and sealed around the pipes, and the top layer (*I*) of concrete is so laid as to leave temporary gaps to give access to the pipes. When no more water is to be removed the pipes are closed by plugs and sealed with an asphalt filling, and the top layer (*I*) is completed by filling in the gaps, the pipes being left in situ.

Treating Slag; Artificial Stone.

374,008.—Dunn, E. P., Ferny Creek, Victoria, Australia. Feb. 23, 1931.

A process for treating molten slag to produce constructional material, aggregates, artificial stone, etc., of uniform composition, consists in removing impurities from the slag by settling and also by oxidation, adding substances, e.g. silica, to combine chemically with the slag, and mechanically incorporating solid materials. The preliminary step of agitating the slag to homogenise it is preferably included. Apparatus shown in *Fig.* 1 includes a settling device and

mixer (2), a desulphurising furnace (8) and a rotary furnace (13). The slag flows into the mixer (2) down a chute (1) and therein passes under a baffle (5) to the outlet (6). Solid particles settle and are removed from an outlet (4). In the desulphurising furnace the slag flows in



TREATING SLAG

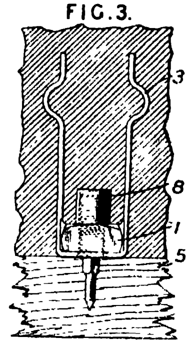
a thin stream over a series of inclined beds (9) where it is exposed to oxidising gases from a firebox (11). The beds (9) are formed with projecting ridges or teeth to break up the stream. The slag then passes to a rotary furnace (13) in which material such as sand is chemically combined with or dissolved in the slag. The sand is preheated in a rotary drum (16), heated by the waste gases from the furnace, and from the drum passes to a hopper (18) and is charged into the furnace by a screw conveyer (19). The furnace is formed with internal ribs (13*c*) adapted to lift and drop the material When the material to be combined is in lump form, a furnace shown in *Fig.* 8 is used instead of the rotary furnace. The lump material is placed in a series of compartments (33) through which the slag flows in succession. Solid materials may be incorporated with the molten slag in a rotary vertical cylinder (21, *Fig.* 4), having transverse bars (22) which may be hollow and cooled from within. The slag and added material are fed into the top of the cylinder and the mixed composition falls into moulds (25). Alternative mixing devices are (*a*) *Fig.* 7, a narrow trough (29) containing a series of star wheels (30) which may be hollow and cooled; (*b*) *Fig.* 6, a fixed cylinder (26) containing a revolving mixing screw (28); and (*c*) a modification of the device shown in *Fig.* 6 having the screw fixed and the cylinder revolving.

Blocks with Inserts for Attaching Fittings.

374,536.—H P. Ford, 57, North Road, West Bridgford, Nottinghamshire March 18, 1931.

Sockets, such as nuts (1), forming means for attaching fittings, are secured in a cast mass, for example a ceiling, wall, or floor, by fixing them temporarily by means of pointed studs (5) driven into the shuttering and then casting the concrete around them In the case of a nut, the stud is screwed at the upper end for subsequent removal from the finished article and a cap or tube (8) of fibre, paper, or the like may be used to cover the projecting upper end and prevent the concrete binding in the threads. The nut

FIG.3.



BLOCKS WITH INSERTS.

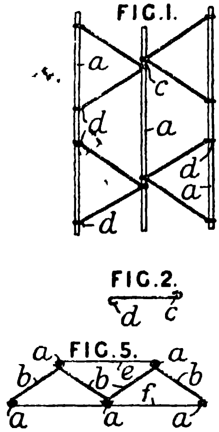
may be surrounded by, and in some cases welded at the sides to, a stirrup (3) curved towards the upper end to embrace one or more reinforcing rods.

Reinforcing Fabric.

374,582.—P. S. Reid, Newlands, Hillside Road, Ewell. April 30, 1931.

In mesh reinforcement the longitudinal members are interconnected by links enabling the fabric to be folded in concertina fashion. The links may be formed at their ends to clip on to the longitudinal

members. In the form shown, the links (*b*) span from one bar to an adjacent bar, and comprise V-shaped links made from a continuous length of wire and turned up at the ends of the limbs to form clips (*d*), and at the junction to form corresponding

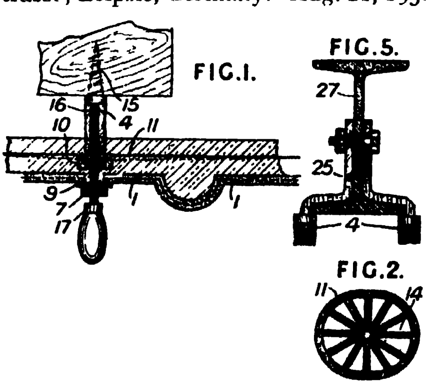


REINFORCING FABRIC

clips (*c*). Where some of the bars (*a*) alternate with others at a different level, additional links (*e*, *f*) may be used to connect the bars on the same level Instead of being V-shape the links may be straight or of any other form and may be turned up to form clips of different shape Radially arranged bars for circular reinforcements are connected by links of different lengths.

Moulding Ceilings In Situ.

376,065.—W. Süssmilch, 40, Grassistrasse, Leipsic, Germany. Aug. 21, 1931.



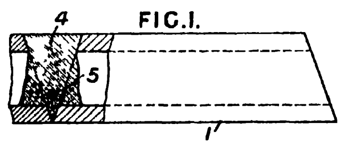
MOULDING CEILINGS IN SITU.

Supports for ceiling shuttering are provided with means for attachment to the floor joists and with axial bores to receive screws having means to support the shuttering. The supports (4) are provided with wooden screw or spiked ends (15) for attachment to wooden joists and have a bore (16) for a screw (17) provided with an adjustable nut and washer (7 and 9) on which rests the shuttering (1), which may be plain or ribbed and formed of reinforced glass. Metal reinforcement for the ceiling may be formed of an open dished metal disc (11) the ribs (14) of which may be corrugated, and this reinforcement is supported on a nut (10) carried by the support (4). For attachment to metal beams (27, *Fig.* 5) the cranked ends (25) of the supports (4) are perforated to receive a bolt.

Hollow Blocks for Concrete Floors.

376,152.—R. Stransky, 12, Mariahilferstrasse, Vienna. Nov. 12, 1931.

A hollow block (1) of the kind having longitudinal lips at its lower edges which abut with the lips of adjacent blocks to



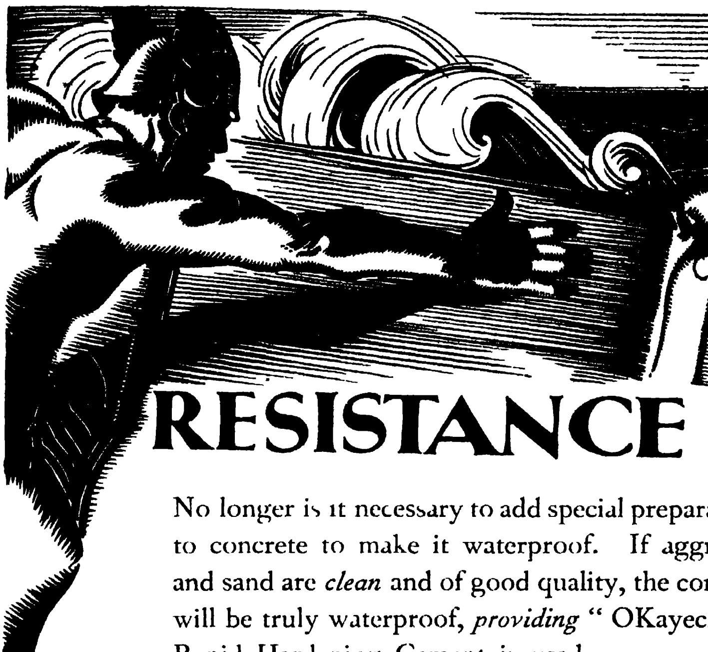
HOLLOW FLOOR BLOCKS

form moulds for concrete joists has its ends chamfered to the full depth of the block to form a transverse V-shaped space (4) into which the concrete enters and thus partly enters the blocks and keys them together. Transverse reinforcing rods (5) are provided.

Floors.

376,228.—L. Navratil, 15, Vilmos csiszar ut, Budapest. Dec. 30, 1930. Void.

In reinforced concrete floors of the kind in which hollow bodies are disposed between reinforced concreted carriers two or more adjoining hollow bodies are arranged in series at right-angles to such carriers between adjacent carriers. As shown in *Fig.* 1, two adjoining hollow bodies (2*a*, 2*b*) are arranged between adjacent reinforced concrete carriers (1), the contacting walls of the bodies and carriers diverg-



RESISTANCE

No longer is it necessary to add special preparations to concrete to make it waterproof. If aggregate and sand are *clean* and of good quality, the concrete will be truly waterproof, *providing* " OKayecrete " Rapid Hardening Cement is used.

" OKayecrete " contains a very high percentage of active material ground to an exceeding fineness, and results in very compact concrete which is absolutely watertight. There is no need, therefore, for you to go to the expense of adding waterproofing agents to your concrete. When constructing tanks, reservoirs and other structures which must be watertight, use " OKayecrete." It is cheaper and better.

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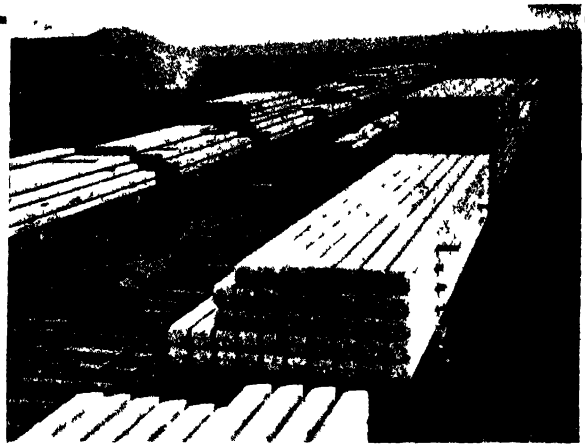

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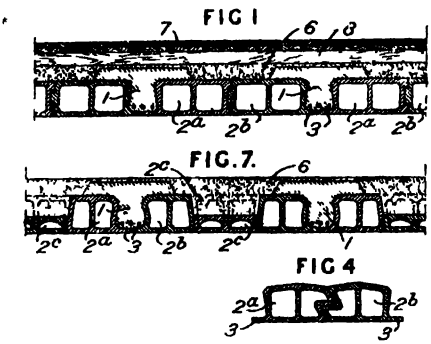
IMMEDIATE DELIVERY. LARGE STOCKS.

Also Shoes, Forks, and other parts.

ing downwardly and the contacting faces of the hollow bodies being provided with tongues and grooves The hollow bodies may be interlocked in various ways or may be cemented together Preferably flanges (3) are provided Fig 7 shows a floor in which three adjoining bodies (*2a*



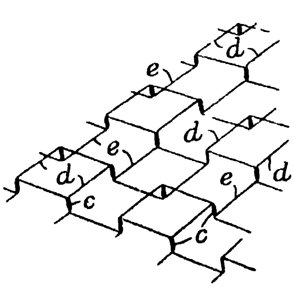
Floors

2b *2c*) are provided the intermediate body (*2c*) being of full or diminished height and having wedge shaped walls adapted to key with the adjoining walls of the bodies (*2a* *2b*) Slag etc filling (6) is provided upon which are superimposed the floor beams (8) and floor boards (7)

Reinforced Concrete

376,580. P S Reid Newlands Hill side Road Ewell Oct 16 1931

A reinforcement particularly a spacing reinforcement for road construction comprises continuous lengths of wire etc having upper and lower parallel portions (*d, e*) and interconnecting vertical parts (*c*) The lengths are assembled as shown with the vertical parts (*c*) bound by wire,



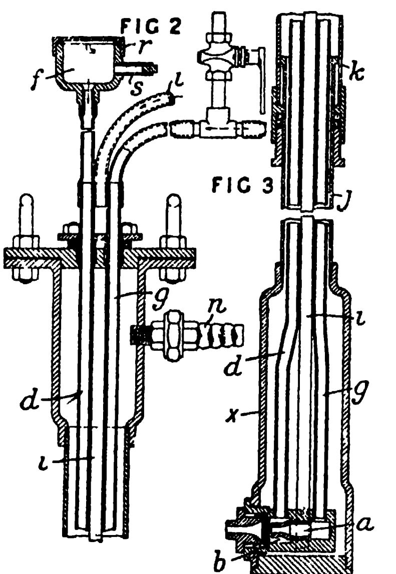
Reinforced Concrete

thus enabling the assemblage to be collapsed for transport The reinforcement may be used for constructing walls

Cement Guns.

382,425.—West, F J West E, and West's Gas Improvement Co Ltd Albion Ironworks Miles Platting Manchester Oct 28, 1931

Vertical retorts are repaired by spraying a liquid medium such as silica clay grouting on to the defective part by an air blast from a nozzle the conduits to



Cement Guns

which are telescopic and are cooled to prevent drying of the repairing medium and to provide a cold air blast for use after spraying to prevent premature setting of the repair The spraying head (*x*) contains an air nozzle (*a*) and an annular space (*b*) to which the repair liquid is supplied through a pipe (*d*) from a container (*f*) the air being fed through a pipe (*g*) Telescopic pipes (*j* *k*) enclose the air and liquid pipes and are supplied with cooling water from a pipe (*l*), an overflow (*n*) being provided The repairing liquid may be supplied under pressure by closing the container (*f*) with a cap (*r*) and connecting a pipe (*s*) to a pressure air supply

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	7	9
Clean Shingle, ¾ in. mesh	,,	6	9
,, ,, ⅜ in. mesh	,,	9	0
Thames ballast	,,	6	9
Broken brick (¾ in.)	,,	10	6

Best British Portland Cement (delivered London area) per ton 46*s*, including non-returnable paper bags; 44*s*. 9*d*, including charge for hire of jute sacks.

Rapid-Hardening Portland Cement . delivered London 7*s*. 6*d*. per ton extra.

"Colorcrete" red and buff rapid-hardening Portland Cement, delivered London 20*s*. per ton extra

"Snowcrete" White Portland Cement, £9 10*s* per ton delivered London, including non-returnable paper bags.

"Super-Cement" Waterproof Cement (including paper bags) per ton 76*s*.

BOARDING FOR SHUTTERING—

		Sawn.	*Wrot.*
1 in.	per square	22*s*. 6*d*.	26*s*. 6*d*.
1¼ in.	,,	28*s*. 6*d*.	32*s*. 6*d*.
1½ in.	,,	34*s*. 0*d*.	40*s*. 0*d*.

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £19 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £18 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	8	6
7/16 in. to ½ in. Rounds	,,	8	9
⅜ in. Rounds	,,	9	0
¼ in. Rounds	,,	10	0

Breeze Slabs per yd. super: 2 in., 1/6; 2½ in, 1/8; 3 in., 2/-; 4 in, 2/4.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—

				s	d
Do.	do.	in foundations	per foot cube	1	3
Do.	do.	in columns	,, ,,	1	5
Do.	do.	in beams	,, ,,	1	7
Do.	do.	in floor slabs 4 in. thick	per yard super	3	8
Do.	do.	in floor slabs 5 in thick	,, ,,	4	7
Do.	do.	in floor slabs 6 in. thick	,, ,,	5	6
Do.	do.	in floor slabs 7 in. thick	,, ,,	6	5
Do.	do.	in walls 6 in. thick	,, ,,	5	6

(Add for hoisting 3s. 6d. per yard cube above ground-floor level. Add for rapid hardening Portland Cement 2s. per yard cube.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	16	0
,, 7/16 in. to ½ in.	,,	15	0
,, 9/16 in. to 2⅞ in.	,,	14	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d*.; ⅜-in rods, 1*d*.; ½-in. rods, 1¼*d*.; ⅝-in. rods, 1½*d*.; ¾-in. rods, 1¾*d*.; ⅞-in. rods, 2*d*.; 1-in. rods, 2½*d*.; 1⅛-in. rods, 3*d*.; 1¼-in. rods, 3½*d*.; 1⅜-in. rods, 4½*d*.; 1½-in. rods, 6*d*.

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d*.; ⅜ in., 2*d*.; ½ in., 2½*d*.; ⅝ in., 3*d*.; ¾ in., 3½*d*.; ⅞ in., 4*d*; 1 in., 4½*d*.; 1¼ in., 6*d*.; 1½ in., 7*d*.

SHUTTERING—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	50	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	46	0
Do. do. in small quantities	per ft. super	0	8
Shuttering and Supports to Stanchions, average 18 in. by 18 in.	per ft. super	0	7½
Do. do. as last, in narrow widths	,, ,,	0	9½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	,, ,,	0	9½
Do. do. as last, in narrow widths	,, ,,	0	10
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/7 per hour; Carpenters working on old shuttering, 1/8; Labourers on building works, 1/2½; Men on mixers and hoists, 1/3½; Bar-benders, 1/3½.

Prospective New Concrete Work.

ALTRINCHAM.—*Bridge.*—Subject to a grant, the Cheshire C.C. proposes to reconstruct the Altrincham canal bridge, at an estimated cost of £24,000.

ARMAGH.—*Sewerage.*—The U.D.C. is to apply for a loan of £30,000 for a sewerage scheme.

BIDEFORD.—*Water Supply.*—The T.C. has applied for sanction to borrow £3,070 for water supply works.

BISHOP AUCKLAND.—*Bathing Pool.*—The U.D.C. proposes to construct a bathing pool near Dam Head.

BLACKPOOL.—*Swimming-Bath.*—The C.B. proposes to construct a swimming-bath for children at the junction of Park Road and Glastonbury Avenue.

BOOTLE.—*Flood Prevention.*—The Bootle T.C. is considering a proposal for the construction of relief sewers.

BOSTON.—*Water Supply.*—The T.C. proposes to construct water-supply works, which includes the construction of pumping stations at Ulceby and Revesby, at an estimated cost of £86,500.

BRIDGWATER.—*Water Supply.*—The R.D.C. has applied for a loan of £35,618 for water-supply works.

CAISTOR.—*Water Supply.*—The R.D.C. is considering a water-supply scheme for the Market Rasen district, estimated to cost £10,000.

CARNLOUGH.—*Sewerage Works.*—The Larne R.D.C. has applied for a loan of £3,000 for sewerage works at Carnlough.

CASTLEFORD.—*Market.*—The U.D.C. proposes to erect a new fish and meat market, estimated to cost £7,500.

CHINGFORD.—*Swimming Pool.*—The T.C. is considering the construction of swimming pool and lido.

CHIPPING NORTON.—*Swimming Pool.*—The T.C. is considering a proposal to construct a swimming pool.

CLAYTON WEST.—*Bridge.*—The U.D.C. proposes to reconstruct Langley bridge.

COALPOOL.—*Sewerage.*—The Wallsall T.C. is to proceed with the Coalpool and Harden sewerage scheme, estimated at £7,721, subject to a loan being sanctioned.

COLESHILL.—*By-pass Road.*—The Warwick C.C. has recommended the construction of a by-pass road at Coleshill, subject to the M.T. making a grant of 60 per cent. towards the estimated cost of £50,000.

DURHAM.—*Roads.*—The Durham C.C. report that the M.T. have made grants of 85 per cent. towards the cost of widening the Coatham Munderville-Darlington section of the Great North Road, estimated at £46,424, and of works on the Rushyford Road section estimated at £68,225.

EASTBOURNE.—*Bathing Pool, etc.*—The Corporation is considering a scheme for the development of Redoubt, including the construction of a bathing pool, estimated to cost £65,000.

EAST BARNET.—*Sewage Disposal.*—It is announced that arrangements are now being completed for the construction of sewage disposal works at East Barnet sewage farm at Brunswick Park. The total cost of the complete scheme is estimated at £135,000.

ENNISKILLEN.—*Swimming Bath.*—The U.D.C. is considering a proposal to construct a swimming bath.

HERTFORDSHIRE.—*Road.*—The C.C. proposes to widen the main road between Hockerill and Stanstead at Bishop's Stortford, at an estimated cost of £11,816.

HODNET.—*Water Supply.*—The Market Drayton R.D.C. is to apply for sanction to borrow £3,600 for a water supply for Hodnet village.

ILKLEY.—*Bathing Pool.*—The U.D.C. is to apply for a loan of £5,000 to construct a bathing pool.

LAMBOURN.—*Water Supply.*—The Hungerford R.D.C. is to prepare a scheme for a water supply for Lambourn and district at an estimated cost of £21,304.

LANCASHIRE.—*By-pass Road.*—Sanction has now been given by the M.T. for the completion of the by-pass road between Preston and Blackpool.

LANGFORD.—*Sewage Disposal.*—It is proposed to construct joint sewage disposal works for the parishes of Langford, Shefford, Henlow and Clifton, at an estimated cost of £60,000.

LEICESTER.—*Abattoir.*—The T.C. is considering the construction of an abattoir.

PORTLAND CEMENT — CONCRETE AND CONSTRUCTIONAL ENGINEERING

'FERROCRETE'

WHEN HIGH EARLY STRENGTH CONCRETE WAS CALLED FOR

on the

UPPER YANGTSZE

From all quarters of the globe reports are received of work of an unusual
on which "Ferrocrete" has been used. Here is one from China.
In order to make a deep mid-stream channel at K'unglingt'an on the Upper Yangtsze
it was found necessary to remove considerable rock formations which were only partly
exposed at dead low water level. It was decided last winter, therefore, to build
reinforced concrete platforms on the rocks in the form of "ships," rising to a height of
11 ft. above local zero level. For this purpose, "Ferrocrete" rapid-hardening Portland
Cement was specified because a saving of time was so important in view of a possible
early rise in the water level. Actually, shot firing was carried out inside the "ships"
within 4 days of pouring the concrete. Although subject to such violent treatment, the
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LINTON.—*Water Supply.* The R.D.C. is to apply for a loan of £ o,000 for a water supply scheme.

LLANDUDNO.—*Sea Defence.* The U.D.C. is considering a scheme for sea defence works on the North Shore.

LLECHRYD.—*Bridge.* The Cardiganshire C.C. has decided to carry out reconstruction work in concrete on the Llechryd bridge over the river Teify, at an estimated cost of £5,000.

LONDON (WOOLWICH).—*Road.* It is reported that work will commence shortly on the Eltham Lee section of the circular road from Woolwich to Battersea, and that the L.C.C. has decided to spend £40,000 on the scheme.

LOWER WOLVERCOTE.—*Bridges.* The Oxfordshire C.C. is considering the proposed construction of two canal bridges and a railway bridge at Lower Wolvercote.

LYDD.—*Concrete Road.* The T.C. has been informed that it is proposed to construct a concrete road from Greatstone to the Pilot, at an estimated cost of £10,000.

MARKET RASEN.—*Water Supply.*—The Caistor R.D.C. is to apply for sanction to proceed with the Market Rasen water supply scheme, estimated to cost £10,000.

MATLOCK —*Bathing Pool* – The U D C proposes to construct a bathing pool

MORECAMBE —*Sewerage* – The T C has decided to adopt a new sewerage scheme which together with improvements to the existing works is estimated to cost £200 000

NEWTOWNBREDA *Sewerage* The Hillsborough R D C is to apply for a loan of £6 030 for the Newtownbreda sewerage scheme

NORTON *Pumping Stations etc* —The U D C is considering a proposal to construct three pumping stations at an estimated cost of £29 150 and a joint sewerage scheme for Malton and Norton

PENZANCE *Bathing Pool* —The T C is to proceed with the construction of a bathing pool at an estimated cost of £14 000

PETERBOROUGH —*Swimming Bath etc* —The T C is to submit to the M H a scheme for the construction of a swimming bath and the erection of a bus station estimated to cost £14 990

PORTOBELLO *Pier etc* —It is reported that a company is being formed to erect a pier and esplanade at Portobello Edinburgh

PULBOROUGH *Bridge* The Worthing M C is considering a proposal to construct a reinforced concrete bridge over the river Arun at Pulborough at an estimated cost of £13 000

SOLWAY FIRTH *Canal* The Carlisle C B C proposes to construct a canal joining Solway Firth with the Tyne

SEVENOAKS *Swimming Bath* —The U D C has applied for sanction to borrow £2 500 for the construction of an open air swimming bath

STONE *Swimming Bath* The U D C proposes to construct a swimming bath at an estimated cost of £1 700

STONEHAVEN *Swimming Pool* The T C is to proceed with the construction of a swimming pool at an estimated cost of £5 500

TRALEE *Concrete Viaduct* The Harbour Board proposes to construct a concrete viaduct at Fenit port subject to a grant being obtained towards the estimated cost of £48 000

TRITLINGTON —*Road Works* — The Northumberland C C has decided subject to a grant to proceed with work on the Great North Road between Tritlington and Felton at an estimated cost £119 933

WATERLOO WITH SEAFORTH — *Swimming Bath* The U D C is proposing the construction of a swimming bath

WORCESTER — *Flood Prevention* —The River Severn Catchment Board proposes to construct flood prevention works in the lower reaches of the river Severn

WORKINGTON —*Swimming Baths* – The T C proposes to construct swimming baths

YIEWSLEY – *Bathing Pool* –The West Drayton U D C has applied for sanction to borrow £2 800 for the construction of an open-air bathing pool

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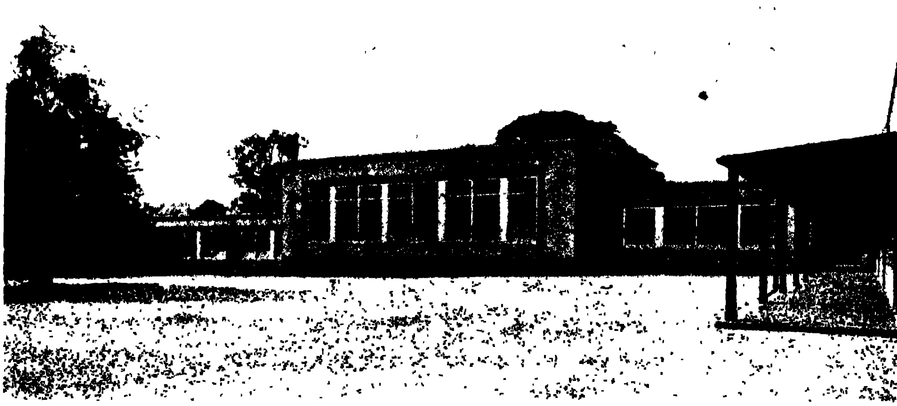

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ECONOMY in cost being essential, reinforced concrete has been employed here to an extent unusual in this type of building, with eminently successful results.

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We illustrate a general view from the playground, with the classrooms and cloakrooms leading from the central Assembly Hall. The administrative block, the only two-storey portion of the building, containing the head-master's room and staff rooms, is also in reinforced concrete. The school gives a delightful impression of airy freshness and light, and is an excellent example of the great value of reinforced concrete in the attainment of a pleasing and efficient structure at a most economical cost.

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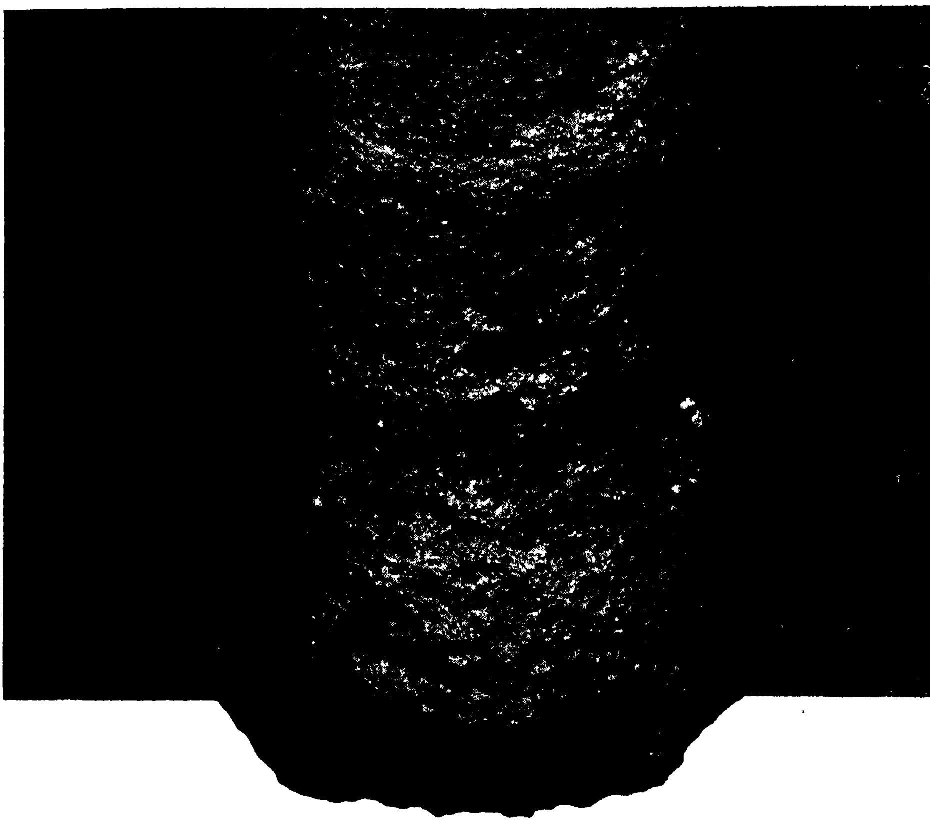

This illustration of a dug-out Franki pile shows at a glance some of its advantages over other forms of piling. Note particularly the massive club foot formed by compressing the surrounding earth until absolute resistance is achieved ; note the rough irregular surface of the pile, giving maximum skin friction throughout its entire length, and the great diameter of the column caused by forcing compressed concrete out beyond the diameter of the piling tube.

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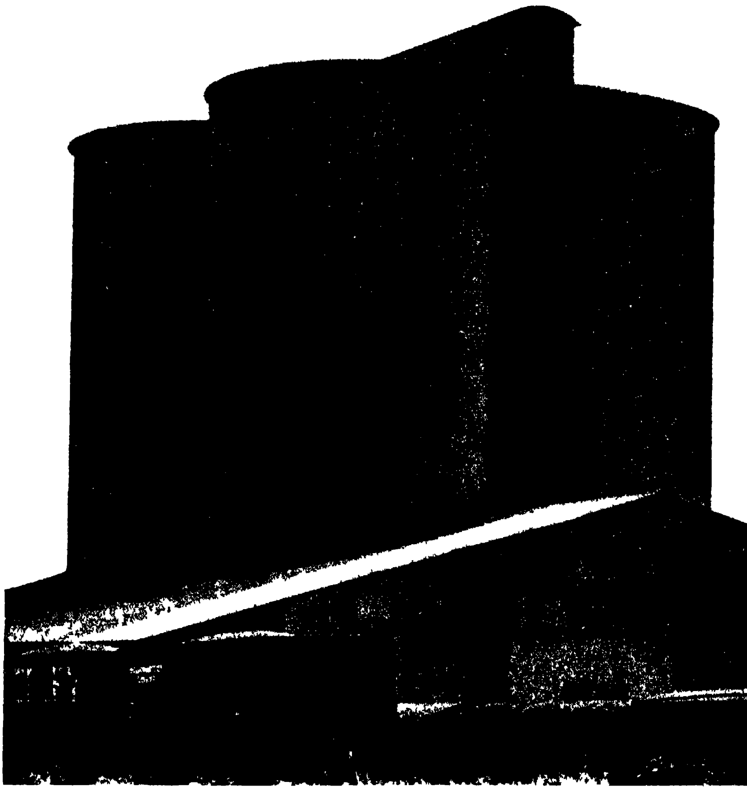
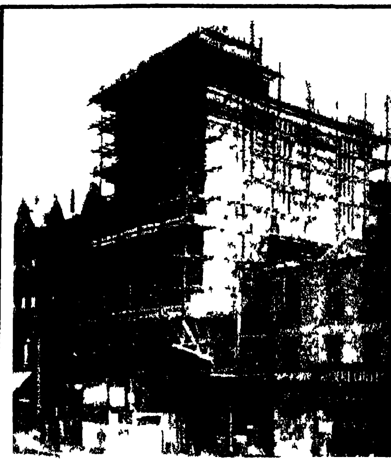
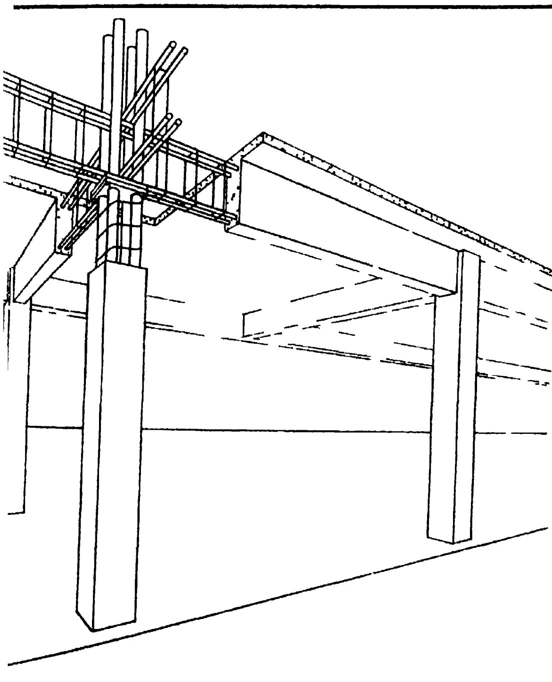


Made possible the erection of this building on an awkward site





We illustrate above Shropshire House London for which "Multiple columns were used The awkward site of this structure involved many difficulties in the design as there is a roadway entrance on one side where the projection of the columns was restricted to 9 in and only three columns could be used Reinforced concrete or steel columns of conventional design 15 ft high would considerably exceed the maximum permissible width of 9 in , and the problem was solved by using "Multiple" columns

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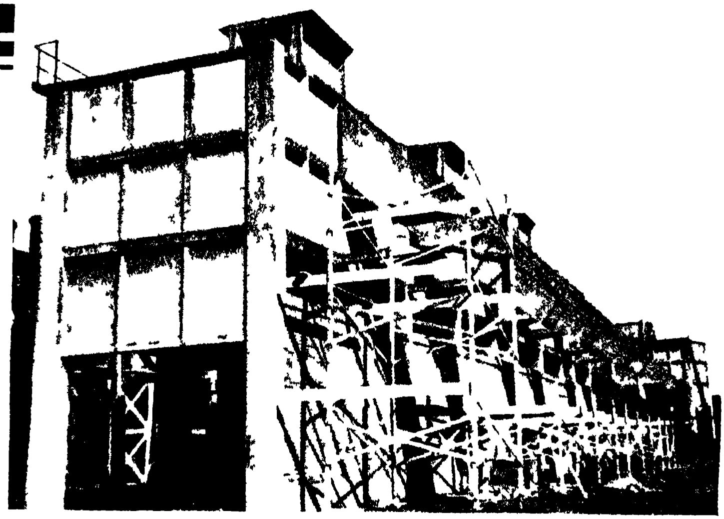

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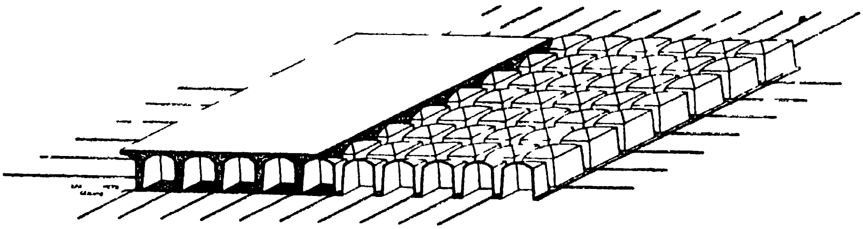

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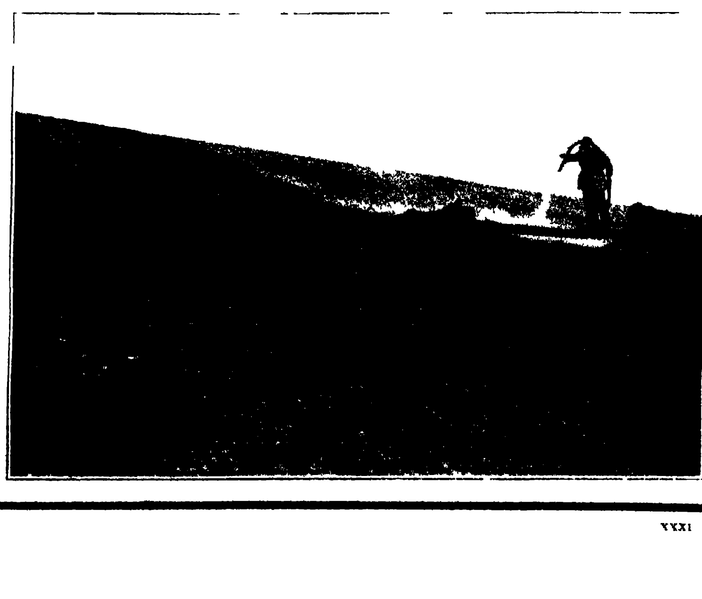

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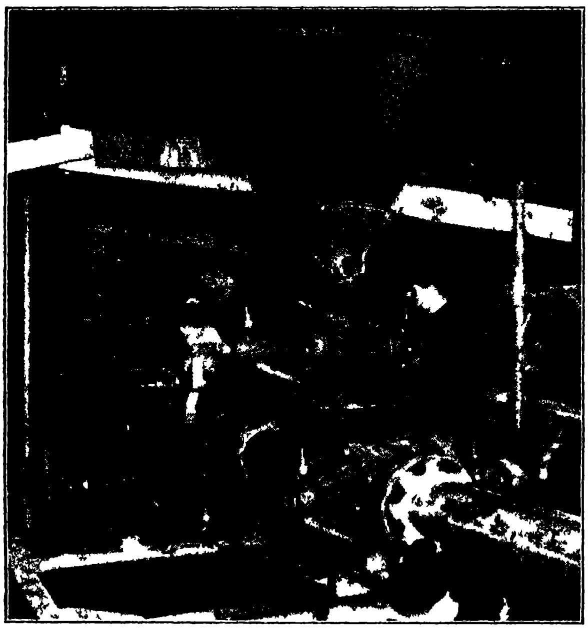

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Volume XXX No. 1 JANUARY 1935

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The London County Council Regulations.

SOME CONCESSIONS GRANTED DURING 1934

DURING the past year reinforced concrete engineers in the London area have welcomed a number of important concessions granted by the London County Council which enable designers to take advantage of the improvements in the quality of Portland cement, the fuller knowledge of methods of making high-strength concrete, and the more accurate estimation of the loads superimposed on structures of varying types. Although it has not yet been found possible to issue new regulations for reinforced concrete, the Council's powers of waiver can be exercised to avoid the stringent requirements of the existing rules and place reinforced concrete design and construction in London on a basis more nearly in accord with that existing outside Great Britain. The waivers which have been granted are a step towards more scientific and therefore more economical design, and their general adoption throughout Great Britain is only a matter of time.

Two important factors in the design of an economical framework to support a given load are the correct estimation of the superimposed load and the maintenance of the weight of the structure or the dead load at the lowest possible level. Insofar as the former is concerned it is clearly in the public interest that the estimate should err on the side of safety, but at the same time the owner of the structure must not be penalised by being compelled to use an unnecessarily high quantity of building material to carry the load. In the past heavy expenditure has often been required to meet the regulations of public authorities. For instance, the Board of Trade rule specifying a horizontal wind pressure of 56 lb. per square foot on railway bridges, which was adopted after the Tay Bridge disaster in 1879, was undoubtedly very severe in all but the most exposed situations, and its adoption has greatly increased the cost of many works. At a later date it was found that the relation between the velocities of the wind and the anemometer cups had been overestimated, but the wasteful effect of the rule continued. In building construction also the tendency in the past was to specify heavy loadings, and it was not until the adoption of the Code of Practice for the use of structural steel that this handicap in design was removed by the London County Council. For reinforced concrete construction new regulations have not yet been issued by this body, but it is now possible to obtain considerable concessions by waiver. In this way the floor loads contained in the Code of Prac-

tice for structural steel may be permitted in reinforced concrete work. Moreover, in rooms to be used for domestic purposes, a loading of 40 lb. per square foot (exclusive of any allowance for partitions) is permitted instead of the 70 lb. required by the existing regulations. All floors must be capable of carrying in any position on an otherwise unloaded floor a concentrated load, in the case of floors of rooms used for domestic purposes, of ½ ton. This alternative concentrated load is to be considered in each case as equally distributed over a floor area of 2 feet 6 inches square. The reactions due to these alternative concentrated loads need not be allowed for in calculating the loads on pillars and foundations.

Instead of the 112 lb. per square foot specified in the 1909 Act a superimposed load of 80 lb. per square foot is permitted on floors used wholly or principally for the purpose of a retail shop, provided that all floor beams are capable of carrying in any position on an otherwise unloaded floor a concentrated load of 2 tons. Office-floor loads are also reduced by waiver from 100 lb. per square foot to 50 lb. per square foot plus 20 lb. per square foot for partitions, subject to (1) all floor constructions between the supporting beams of the skeleton framework being calculated to support a superimposed load of 100 lb. per square foot, and (2) the additional allowance of 20 lb. per square foot for partitions being increased if necessary.

In calculating the total loads to be carried on foundations, pillars, and walls, the reductions in the superimposed loads (other than those on the roof and top floor) have been doubled. This increased allowance is to some extent counterbalanced by the reductions made in the floor loads.

Reference has already been made to the importance of a small dead load in economical construction. In large structures the stresses due to dead load generally bear a high proportion to the total stresses on account of the great weight of the members. Increased working stresses are helpful in reducing dead weight and when combined with a suitable design have an important effect on the total cost of a structure. Their relative importance, however, depends to some degree on the cost of materials and formwork. Waivers have been granted by which a maximum compressive stress of 750 lb. per square inch is permitted in 1 : 2 . 4 concrete, subject to an ultimate compressive resistance of 3,000 lb. at four months, and maximum stresses of 1,000 lb. and 930 lb. in 1 : 1 : 2 concrete, subject to an ultimate strength of 4,000 lb. at four months.

Under the existing regulations no provision is made for flat-slab or mushroom floor construction, a type of structure which has been very widely adopted outside Great Britain for offices and warehouses. Owing partly to the lack of official recognition flat-slab construction in this country has been almost entirely confined to reservoir roofs and a certain class of piled foundation, but the grant of waivers by the London County Council for structures of this type will probably give rise to its more frequent use. In the November issue of CONCRETE AND CONSTRUCTIONAL ENGINEERING a flat-slab warehouse was described, which is believed to be the first of its kind erected in London. We understand that a waiver has been granted for a building with flat-slab floors and roof in which a 1 : 1½ : 2½ concrete with a compressive strength of 5,800 lb. per square inch will be used under maximum compressive flexural stresses of 1,450 lb. per square inch and the tensile stress in the steel will be 18,000 lb. per square inch.

Ground-water Lowering and Chemical Consolidation of Foundations.

By H. J. B. HARDING, B.Sc., A.M.Inst.C.E.

A LARGE rebuilding scheme is in progress for Messrs. Bentalls at Kingston which will provide a steel frame building faced with brick and stone extending for 626 ft. along Wood Street and 230 ft. facing Clarence Street. Sir Aston Webb & Son are the architects for the whole scheme of reconstruction. The first two portions in Wood Street were completed by Messrs. John Mowlem & Co., Ltd., in 1931 and 1932. The building is founded on waterlogged ballast and a concrete raft tanked with asphalt in which the new stanchion grillages are bedded carries the building. Considerable difficulty was encountered in building the first two sections owing to the quantity of water in the fine sandy ballast, as the river Thames is only 100 yd. to the west of the site.

The extension described in this article is on an important corner site at Wood Street and Clarence Street, at present occupied by Messrs. Bentalls' original premises (*Fig.* 2) built in the last century, consisting of three buildings of poor quality brickwork strengthened in parts with steel beams and with a number of steel and cast iron columns.

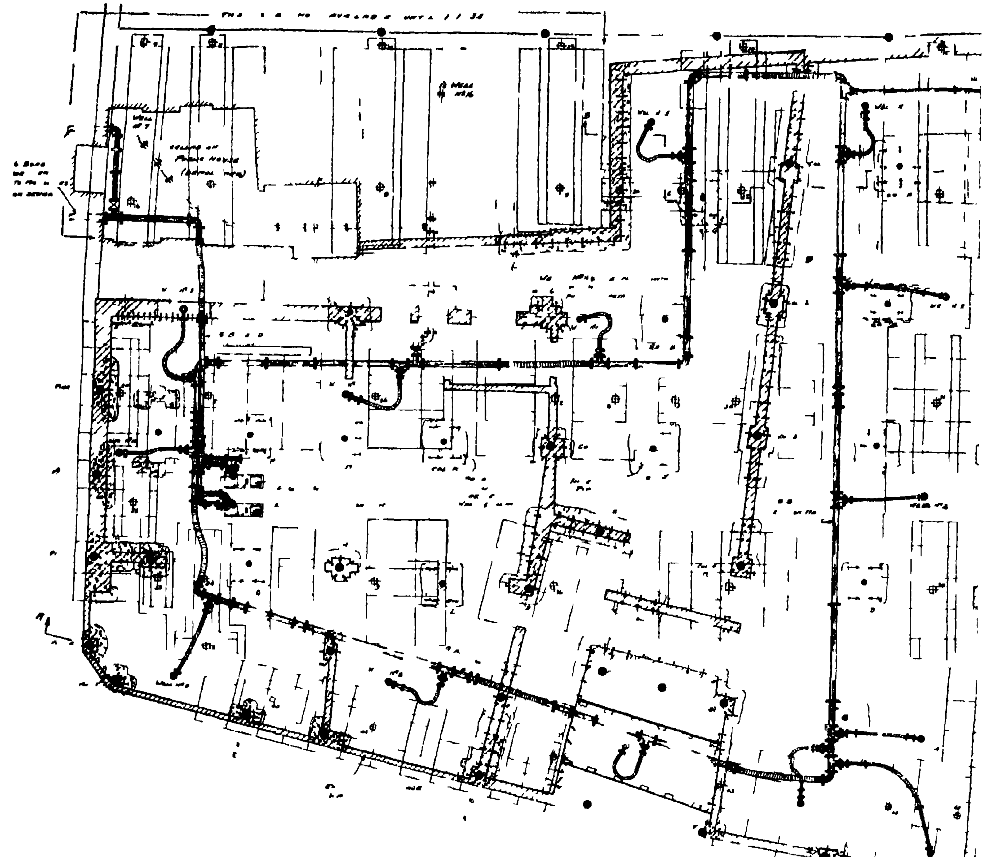
The scheme adopted for the reconstruction of this portion was dictated by economic considerations. The premises were in full occupation and it was necessary for the shopping floors to be undisturbed during the heavy Autumn and Christmas trade, and for rebuilding to be carried out during the quieter Spring season to be in readiness for the Summer trade.

Mr. B. I. Hurst, M.Inst.C.E., the consulting engineer for the rebuilding, devised a method of completing the stanchion foundations and basement retaining wall for the new building without disturbing the foundations of the existing structure. This was made possible by working in the cellars of the existing premises. The foundations of these had been originally built at about + 16.0 O.D., which was the water level in the ballast. In some cases the headroom was as little as 7 ft., the cellars being used only for storage purposes. In the new design the concrete raft is founded at + 10.75 O.D. with basement floor level of + 14.75 O.D., which provides a commodious lower ground floor space of considerable trade value.

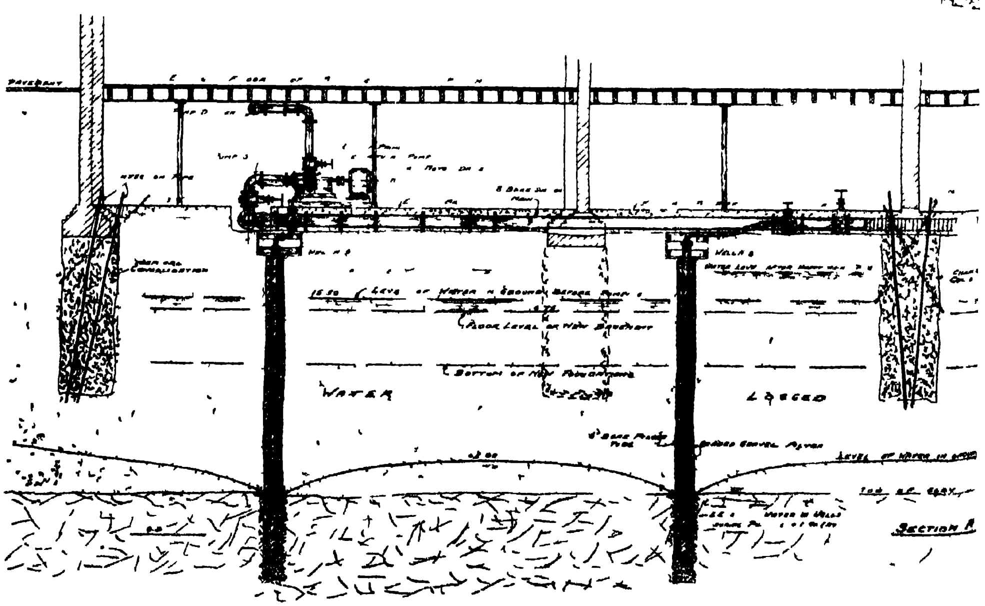
It was found that if the retaining wall and foundations could be constructed in the cellars before demolition, four months would be saved in the time between demolition and rebuilding. For a rapidly-growing store this represents a saving of many thousands of pounds in trade, and prevents less obvious losses due to lack of service and disorganization. The programme allows for two or three floors to be returned to use within eleven weeks of closing.

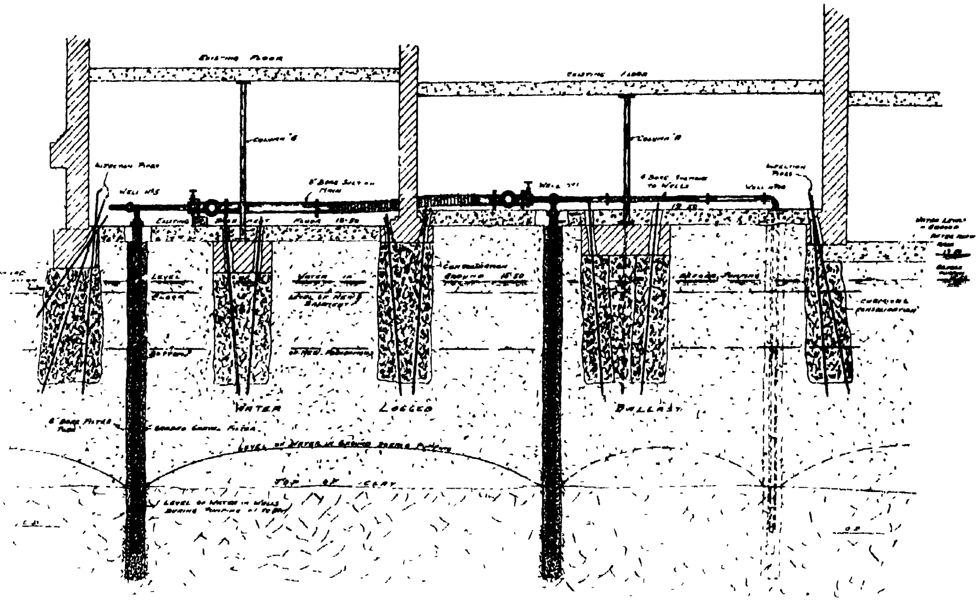
Design of Foundations.

A foundation raft of concrete is being laid all over the site at + 10.75 O.D., and the raft is tanked with three coats of asphalt laid on the bottom 6 in. of concrete. The basement floor level is + 14.75 O.D. and in order to gain unobstructed space the bottom grillage joists are embedded in the bottom of the raft and chases left in the concrete to take the top tier of grillages. The retain-

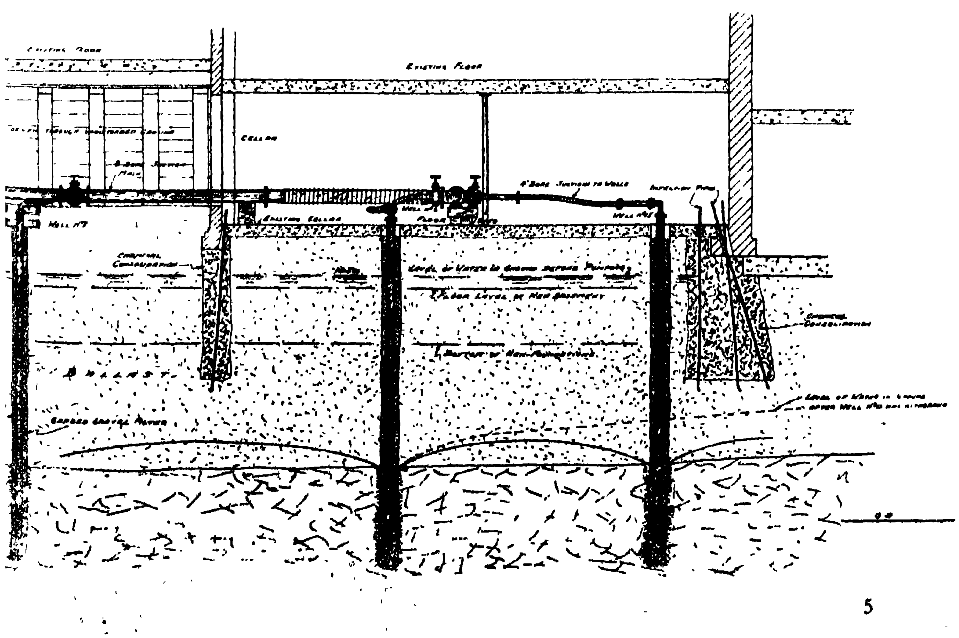


PLAN




Existing Floor
Existing Floor
Water
Logged
Ballast
Top of Clay

SECTION B B


Existing Floor
Cellar
Ballast

ing wall is formed by underpinning the existing walls, and at the same time bringing up the asphalt in a vertical dampcourse. After demolition of the existing premises the inner portion of underpinning will be removed and the existing retaining wall, with its underpinning and asphalt face, will be supported by a reinforced concrete slab spanning vertically between foundations and ground floor.

The heaviest weights on the column bases (*Fig.* 1) amounted to 165 tons,

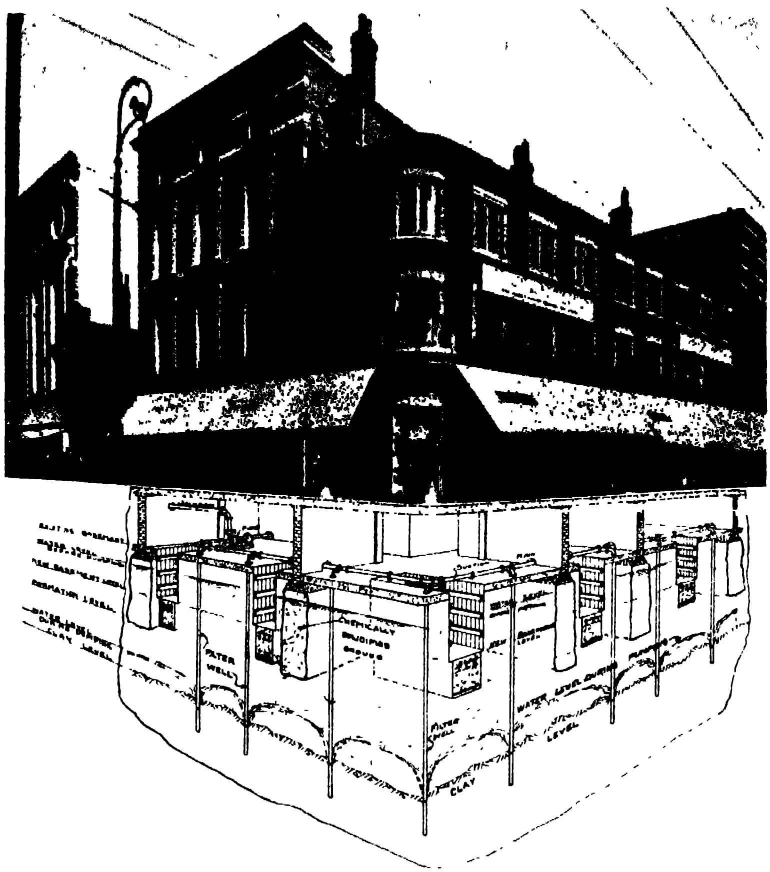


Fig. 2. View of Premises while work, as shown in the diagram, was in progress.

while the brick walls carried loads varying from 4 tons per foot run to 1 ton per foot run. These weights are not large, but the buildings occupying this area had originally been built in piecemeal fashion and at different times, and needed very careful support during the work as they were also crowded with shoppers and were faced with large plate-glass windows.

Chemical Consolidation.

The contractors for the foundations of the present extension are Messrs. Edmund Nuttall Sons & Co. and John Mowlem & Co. (Joint), Ltd., and work

was commenced on August 24 1934 Mr B T Hurst instructed the contractors to employ the Joosten chemical consolidation process, for which they hold the English rights as a means of strengthening the foundations before weakening them by excavation In this process injection pipes are driven into the ground and the ballast and sand are consolidated into a form of artificial sandstone by successive injections of two chemicals The first chemical is injected in measured quantities as the pipe is driven into the ground, and the second chemical is injected as the pipe is withdrawn On meeting the chemicals combine almost immediately to form a solidified mass with a crushing strength of about 30 tons per square foot

Fig 3

Before any excavation work was commenced pipes were driven beneath the walls and columns as shown on cross sections *AA* and *BB* on *Figs* 1 and 7 and the ballast and sand solidified to a depth of 2 ft below the new excavation level In this way the weight of the building was carried on firm supports to a depth lower than the excavation before any weakening took place, thereby avoiding the risk of drawing ground from beneath the walls *Fig* 3 shows one of the chemical plants in operation and *Fig* 4 shows chemically solidified ballast exposed in a foundation pit Chases cut by air hammers can be seen in the consolidated material

The ground under the footings was very dirty ballast, which became cleaner about 2 ft lower down Several beds of fine sand occurred in patches 1 ft to

2 ft. thick but not extending over the whole site. The sand and ballast were found to be solidified very evenly, while the dirty ballast immediately under the walls was also solidified and found to be tightened up under the footings. The fact that column and wall loads were carried down below digging level on consolidated ballast made shoring within the shopping floors above unnecessary.

Ground-water Lowering.

Any method of pumping to be adopted necessitated considerable care in order to prevent movement of sand from beneath the newly-constructed raft

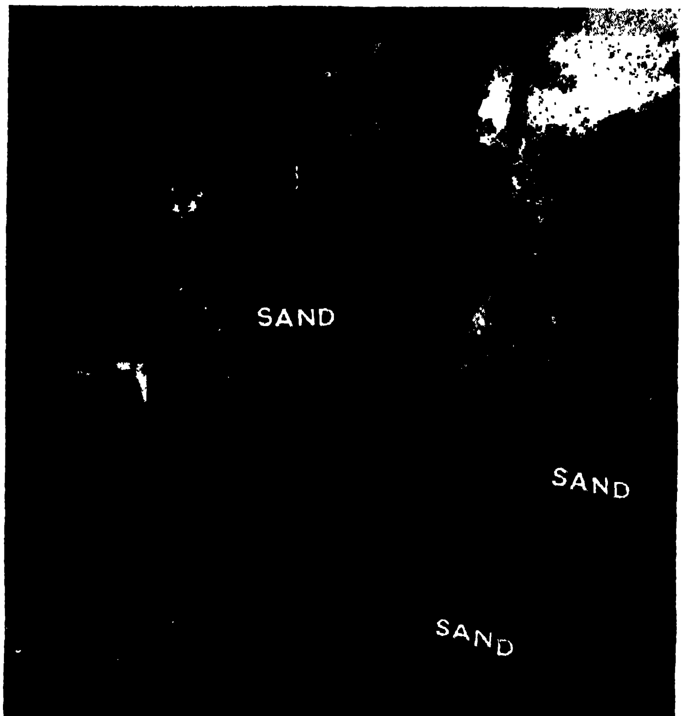


Fig. 4.

foundation of the first extension on the north of the site, as well as the existing buildings on the east. The ground water had given considerable trouble when the first extension was built and, although a number of sumps were sunk and the water conducted to them by subdrains, it was found necessary when underpinning to place the bottom layer in bags of aluminous cement concrete, as the water proved to be flowing very freely and the sand bottom " boiled " up in the trenches.

For the new extensions the contractors submitted a scheme for a shallow-well system of lowering the ground water. This was adopted by the engineer and architects and is illustrated on the drawing. The system adopted is not unlike the deep-well ground-water lowering which was carried out by the same

Fig. 5. Part of Basement shown in Fig. 6 before Excavation.

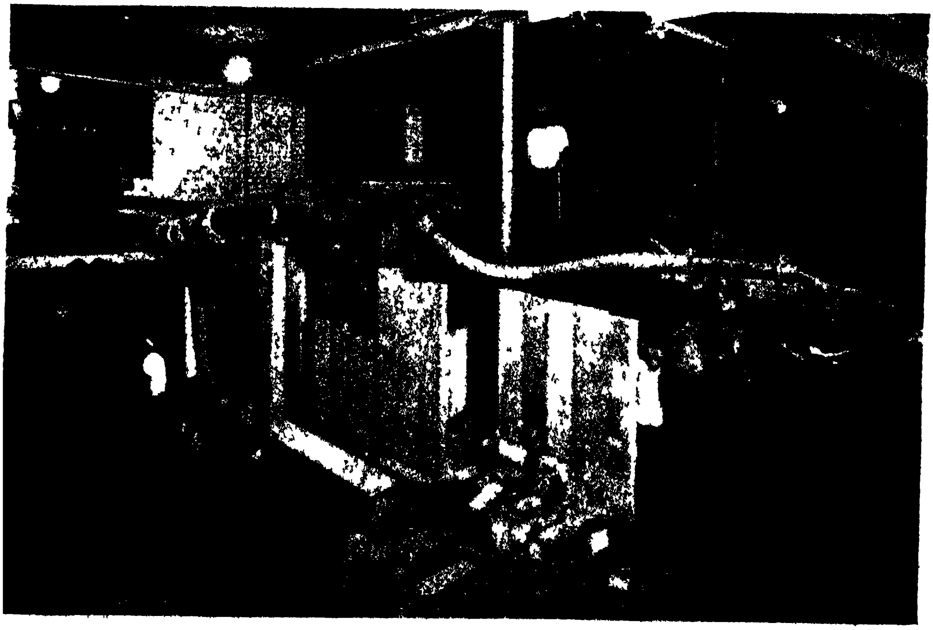


Fig. 6.—New Column Foundations and Wells.

company in conjunction with Messrs Siemens Schuckert (Great Britain) Ltd for Sir Lindsay Parkinson & Co Ltd on the entrance to the new Grimsby fish dock However in this case a number of shallow depth wells are connected to one pump on the surface which serves all the wells

In order to lower the water level in the ground considerably below formation level and to prevent sand being drawn from the ground a series of fifteen

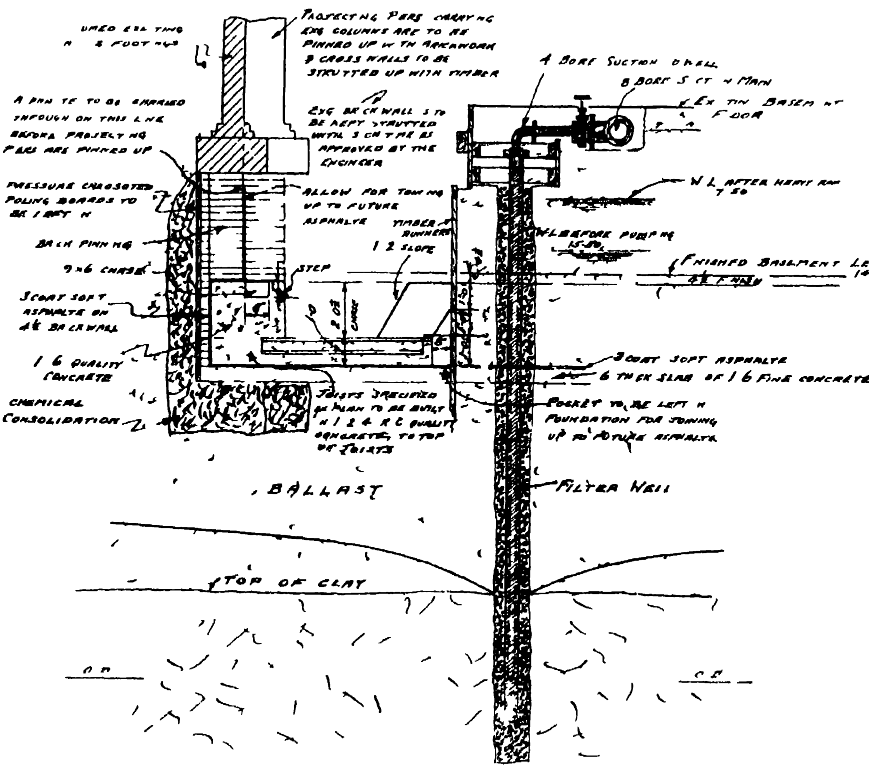


Fig 7 Typical section through underpinning of front walls, which also incorporate future stanchion foundations, also showing consolidation under pier and filter well

filter wells was sunk in the cellars at about 30 ft centres The wells were bored 12 in in diameter and were then fitted with 6 in diameter filter tubes wrapped with two coverings of fine mesh The space between the boring tube and the filter was filled with special gravel graded to suit the ground met with in the boreholes, so that no sand could be drawn into the well The boring tube was then withdrawn Four-inch diameter suction tubes of wrought iron barrel were installed in each well and coupled to an 8 in diameter ring main (*Figs* 5 and 6) which was laid round the site and led to a 6-in diameter self-priming centrifugal pump driven by a 23 h p electric motor

The spacing of the wells and the amount of water to be handled are decided from experience This system of shallow well pumping has been used to a great extent in the United States, Germany and the Low Countries, and a consider-

able amount of experience is available of such work in various conditions. The spacing of the wells depends upon the suction head available, the nature of the ground, the positions possible on the site, and the position of the "draw-down" curve with relation to the formation level. When pumping is carried out from several points simultaneously, the water table in the ground is lowered very sharply at each well—this is known as the "draw-down." The water between wells assumes a curve whose shape depends upon the nature of the ground and the depth of the "draw-down." If an additional well is put into operation midway between two existing wells it has the effect of forming two "draw-down" curves between the three wells and so very considerably reduces the original summit of the water table. Where wells are used for water supplies this effect is known as "interference between wells" as it has the effect of reducing the output per well without any great increase in total supply obtained. However, for ground-water-lowering purposes the reverse holds good and the wells must be located so as to obtain maximum economic interference in order to produce a considerable lowering of the ground water without pumping an excess of water. In this case the summit of the "draw-down" curve had to be well below + 10·75 O.D., which was only 7 ft. above clay level.

The water level in the ground before pumping commenced stood at about + 15·50 O.D., but rose as high as + 17·50 after heavy rain. While the installation was in progress a temporary test was made with a 3-in. Holden & Brook pump set directly over one well. The results were as follows

In the well, water level	+ 5·00
9 ft. away ,, ,,	+ 12·25
32 ,, ,, ,, ,,	+ 14·25
44 ,, ,, ,, ,,	+ 14·50
Original water level at this date	+ 16·10

With all fifteen wells working the water table became much flatter than for a single well, and is shown on the drawing.

The quantity to be pumped and approximate "draw-down" curves can be deduced from empirical formulæ such as those of Slichter or Kyrilies-Sichardt. In this case a coefficient of porosity for ballast was chosen of K = 0·002 metres per second, and the quantity to be pumped was estimated as approximately 480 gallons per minute. At the commencement of pumping at least 700 gallons per minute were pumped, but after the water table had been reduced down to the clay only about 350 to 400 gallons per minute were delivered. The discharge increased after heavy rain.

The wells were bored 30 ft. below pump level to ensure that the suction pipe was always submerged below any possible suction head. The wells consequently penetrated into the clay, which was found to slope towards the river from + 3·50 O.D. to + 2·50 O.D. The filter gravel around the filter tubes formed collecting areas, and the average level in the wells during pumping was + 1·50 O.D. or 18 in. below clay level with a suction head of 22 ft., the axis of the pump being set at + 23·50 O.D. The curve of depression of the water became fairly flat and, by measuring the level in perforated tubes driven into the ground between wells, was found to rise to + 5·00 O.D.

Duplicate pumps were provided to ensure continuous pumping throughout, and conditions of installation were not easy. The wells had to be located to

miss the new stanchion bases and had to be at some distance from the ring main, so flexible hoses (*Figs* 5 and 6) were used in connecting the iron suction pipes to the ring main In normal conditions of an open site it is customary for wells to be close to the suction main and coupled directly to it with a single bend, while the pumping main is located outside the work where possible As there was only 7 ft of headroom in places, a special boring gear was used It was necessary for the filter tubes and suction pipes to be in short lengths in order to insert them in the wells As the cellars were on three different levels varying by 3 ft the suction main was sunk into the floor in one cellar and packed up in another

In the north east corner an area of 34 ft by 70 ft was not available until November 1 1934 As soon as this area was handed over to the contractors two additional wells, Nos 16 and 17, were sunk in the positions shown and wells Nos 11 and 12 were taken out of commission Wells Nos 16 and 17 were coupled to a tee on the ring main which had been provided for this purpose The water level in well No 12 in the centre of the site stood at about + 5 17 O D when the level in No 5 and other pumping wells was at + 1 50 clay level at No 12 being about + 3 50 The wells marked Nos 14 and 15 were sunk as a precaution in case Nos 1 and 5 had not sufficient range to reach the corners of the building However, after pumping had reached a definite level it was found on closing the valves that the water level in Nos 14 and 15 rose to + 7 0 O D which was 3 ft 9 in below formation level As they were no longer required filter tubes Nos 14 and 15 were then withdrawn Before extracting them the wooden plugs which close the bottom of the tubes were driven out and the tubes filled with sand The filters were then pulled out of the ground by means of a winch, and the sand filling prevented voids being left in the ground

When the filter tubes are set in position short lengths of 9 in flanged pipe are threaded on them before coupling up the suction pipes As excavation proceeds these pipes are slid down to formation level and concreted in the floor A band of asphalt is placed round them and the dampcourse joined on to this band When the time comes to draw the wells these pipes will be filled with concrete and closed with a blank flange

The discharge from the pumps is carried into two manholes leading to a high level sewer No sand has been taken out of the ground in pumping The wells do not interfere with the work and there are no open sumps to occupy the ground while the pumping sets are on a dry floor and need only one attendant The contrast between the results from this pumping installation and from the sumps and subdrains on the previous site is remarkable The extraction of water from a number of places with a good suction head lowers the water over the whole area at once This enables work to proceed at many points simultaneously as if on a dry site, without the necessity of digging sumps close to the work on hand

This combination of the chemical consolidation and ground-water-lowering processes has been the means of greatly increasing the speed and ensuring the security of the work, and has also protected the old and new buildings adjoining the site The work would have been attended by grave risk if pumping had been carried out in ordinary open sumps.

The Prevention of Pattern Staining on Plaster Ceilings.

BY R. H. EVANS, M.Sc., Ph.D., A.M.I.Mech.E.

THE disfigurement of plaster ceilings by regular dark stains, showing the general lines of the supporting structure, now receives attention by architects and contractors. In the case of hollow reinforced concrete floors of the beam and tile or other construction with air cavities the stains appear on the plaster ceilings in the course of eight to twelve months and soon afterwards become a source of annoyance. An example of such a ceiling is shown in *Fig.* 1, in which the darker stains form a pattern of the reinforced concrete beams.

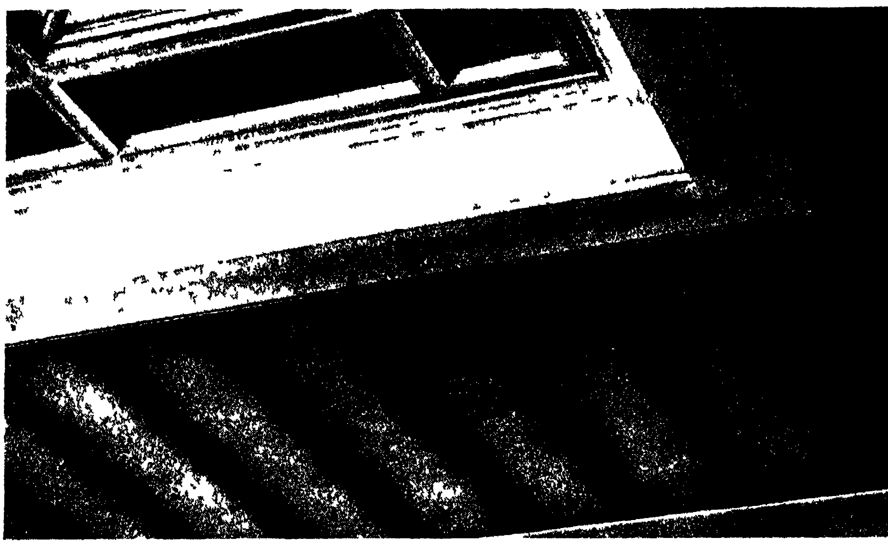


Fig. 1.

The experiments described in this article were carried out to prevent the formation of pattern staining, particularly in plaster ceilings carried by reinforced concrete floors. Stains are caused by the difference in thermal conductivity over different parts of the ceiling with consequent slight differences in temperature. An effective method of eliminating these temperature differences would therefore prevent the formation of a pattern of the beam and slab floor, as the thermal precipitation of the particles of dust and smoke would then take place at the same rate over the whole of the ceiling.

Tests were made on full size samples of many types of reinforced concrete floors complete with plaster ceilings. The building was 15 ft. 4 in. by 4 ft. and 6 ft. 6 in. high and had two windows and one door. Two steam pipes, 10 ft. long placed in the middle of the building, provided the necessary heat, and in order to ensure a uniform temperature on the roof of the building so that each floor would be subjected to the same outside temperature, the roof was covered with ½ in. of asphalt and about 2 in. of water was maintained continually on

the roof Washed smoke particles were introduced this method proving to be a simple and effective means of investigating pattern staining in one day or less

The results discussed here are those of the last two tests which were made on eight floor samples with different methods of carrying the plaster ceilings A section of the floors experimented upon in the first of these tests is given in *Fig 2* while additional data concerning the construction of the slabs are given in *Table* 1

TABLE 1

No 1	Hollow tile floor with 2 in of concrete above top of tile
No 2	Aerodome floor with 2 in of Heraklith slabs under the domes
No 3	Aerodome floor with 1 in wood fillets fastened to the beams and ½ in felting between Ribmet and timber
No 4	Aerodome floor having 1 in wood fillets fastened to beams and Ribmet direct on timber
No 5	Aerodome floor having the expanded metal assembled with the dome so that the concrete and plaster are in contact

During this test the variation of temperature along the different beams and slabs was recorded and two such sets of observations are given in *Table 2* the letters referring to the position of the thermometers with respect to *Fig 2* In each case the thermometer was placed at the middle of the beam or slab *Table 2* shows that comparatively small differences in temperature are sufficient to produce a pattern of the beams and slabs

TABLE 2

Temperature (Deg C)

No 1		No 2		No 3	No 4	No 5			
A	B	C	D	E	F	G	H	Inside	Outside
Beam	Slab	Slab	Beam	Beam	Beam	Slab	Beam		
20.2	20.8	25.0	24.[illegible]	23.3	22.5	21.1	19.1	25.0	12.2
16.3	17.0	21.8	21.4	20.1	19.6	17.5	15.3	22.8	−1.2

The results obtained from the first test show that floors Nos 1 and 5 are liable to have stains at an early stage, floor No 5 showing stains a little earlier than No 1 This agrees with the recorded temperature differences between the respective slabs and beams in *Table 2* Floor No 2 with the Heraklith slabs below the domes was the next to show stains or markings, but the markings were fainter and more irregular and formed no definite pattern The effect of the Heraklith slab and of any other porous material is to diminish the temperature variations under the slab and to eliminate sudden changes of temperature Floors Nos 3 and 4, with the Ribmet ceiling, afterwards showed the characteristic markings, only the darker stains now appeared on the plaster between the beams and not on the plaster under the beams An examination of the

temperatures in *Table* 2 explains this change in the lines of the pattern. Thus floors Nos. 3 and 4 can only be compared with No. 5 in order to obtain an estimate of the slab temperatures of Nos. 3 and 4, for in No. 2 the Heraklith slab acts as a thermal insulator and in No. 1 the hollow tile acts as a conductor. The beam temperatures at E and F are, in floors Nos. 3 and 4 respectively, in each case higher than the slab temperature estimated from floor No. 5, so that greater precipitation of the smoke particles would now occur between the beams and not under the beams. The higher temperatures in the case of beams E and F are due to the insulating influence of the wood fillets, the latter being better heat insulators than the air cavities.

These test results so far show that none of the floors illustrated in *Fig.* 2 is effective in preventing the formation of pattern staining and that felts are of little use in this respect. An effective method of eliminating this trouble in practice should neither be expensive nor call for precise workmanship. Con-

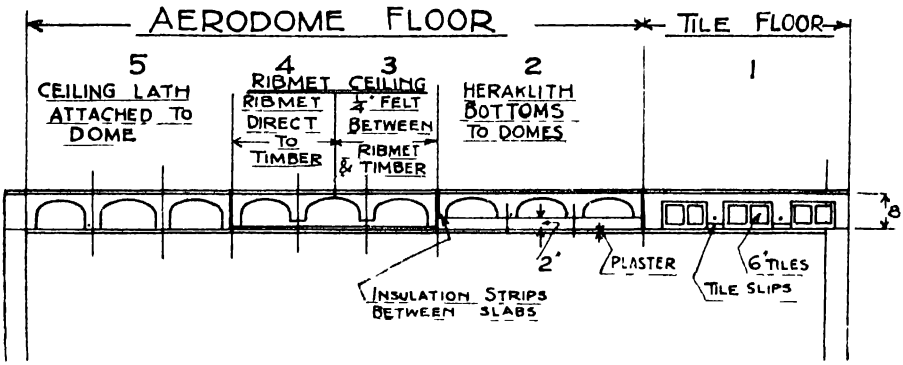


Fig. 2.

sequently a second test was made on five more floors detailed in *Fig.* 3, and in which floors Nos. 1 and 5 of the first test have been again included for the purpose of comparing the results of the two tests. Photographs of the plaster ceilings after the second test are shown in *Fig.* 4. Floors Nos. 1 and 5 naturally show the usual disfigurement, while floor No. 2 with ½ in. thickness of fibre board as a heat insulator is clean and shows no discoloration. Floor No. 3 with the suspended ceiling shows no pattern but is uniformly darker or dirtier than No. 2. This would be expected as the air gap between the floor and the suspended ceiling would maintain the ceiling at a uniform temperature but at not such a high temperature as the fibre board in floor No. 2. In No. 4 timber battens were fixed to the beams and the ceiling was carried by Ribmet, the ribs being fixed transversely to the timber battens. In addition one bay was ventilated while the other was left unventilated as in *Fig.* 4. The photographs show that the ventilated bay is considerably cleaner than the unventilated bay, although the latter does not show any pattern. There is sensibly no difference, from the point of view of cleanliness, between the suspended ceiling method and the unventilated bay in floor No. 4 with wooden battens and Ribmet. This is not surprising as in each case there is an air gap between the plaster ceiling

and the concrete floor, the Ribmet in floor No 4 only making point contact across the bottom of each beam

It is significant from the second test (*Fig* 4) that ventilation of the air space between the ceiling and the flooring by ventilators in the ceiling results in considerably less precipitation of smoke particles The suspended ceiling alone eliminates pattern staining but the provision of ventilators produces a much cleaner ceiling due to the higher ceiling temperature From *Fig* 4 it is clear that ½ in of fibre wood as a thermal insulator is effective and produces as clean a ceiling as the suspended ceiling with ventilators or air gratings Care, however would have to be taken in overlapping all the joints in the fibre wood otherwise markings would appear at these joints in course of time The sus-

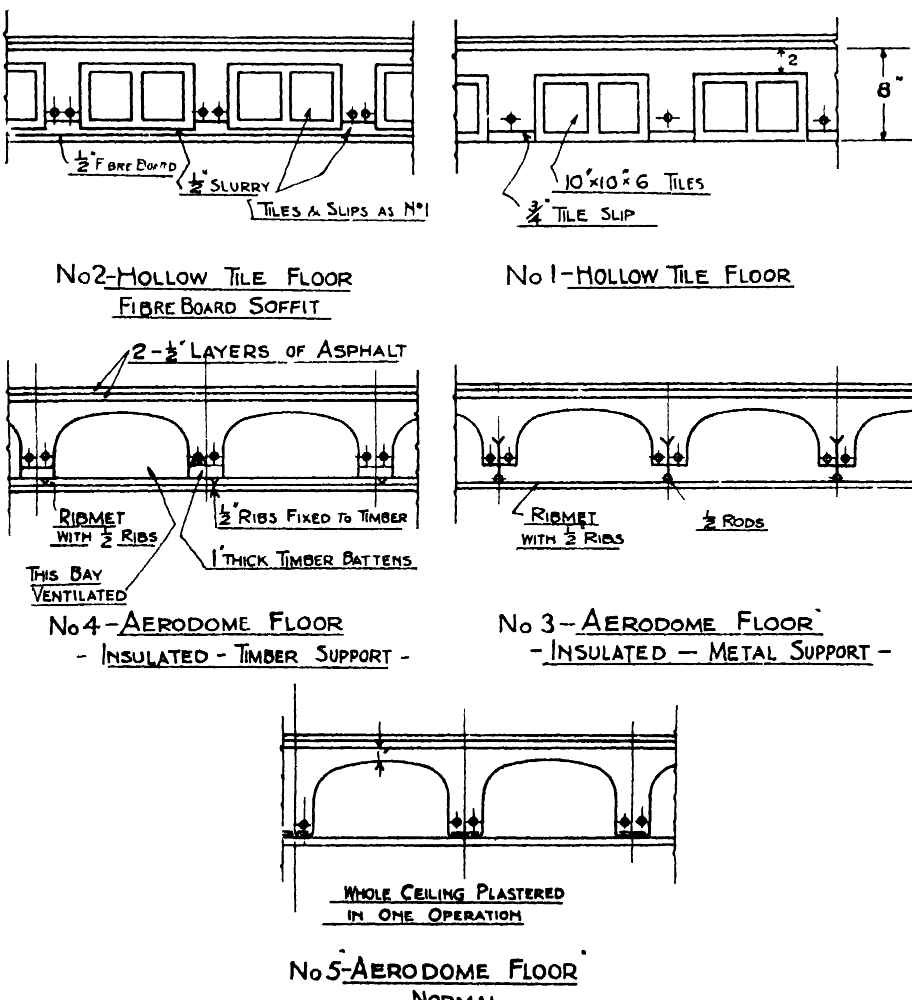


Fig. 3.

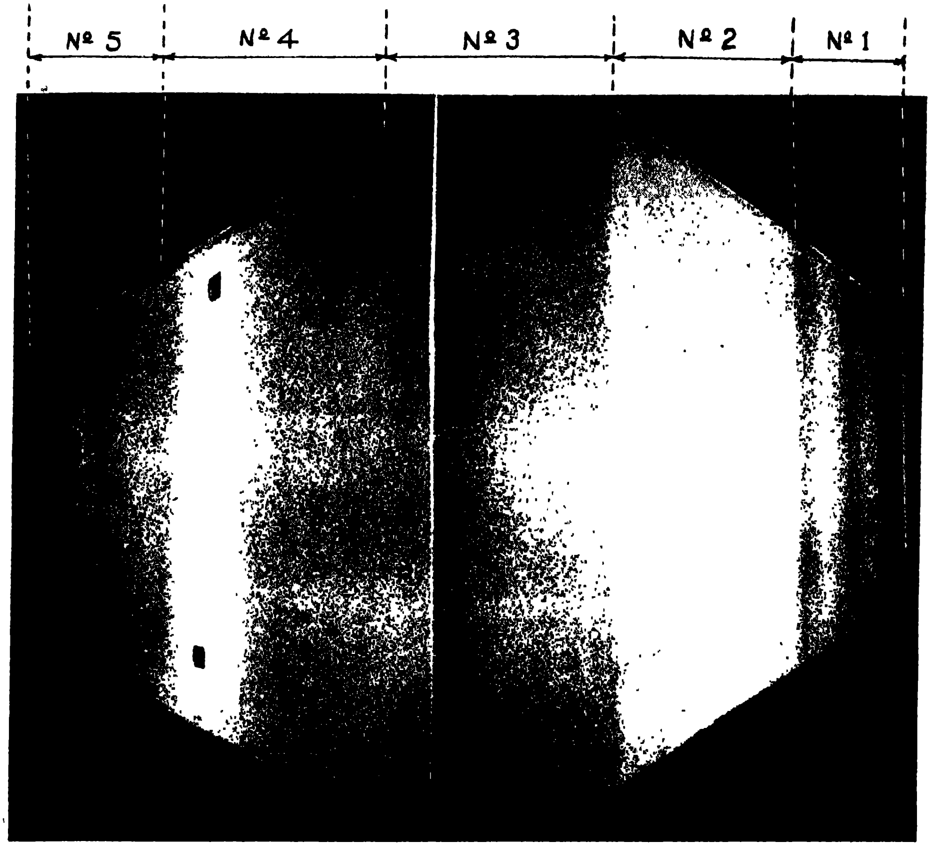


Fig. 4.

pended ceiling with air gratings or ventilators appears to be a simple and economical method of eliminating pattern staining. It may be readily adopted with both the direct and indirect methods of heating buildings in which the question of appearance and the cost of redecoration are important, and it may be applied to any form of reinforced concrete floor having air cavities.

The writer wishes to express his gratitude to Sir Edwin Airey for suggesting and granting facilities to carry out the experiments described in this article, and to Captain J. B. Lawson for his very able and enthusiastic assistance.

Reinforced Concrete Regulations.

In addition to the modifications of its Regulations for Reinforced Concrete given in our issue for December 1934, the London County Council has allowed the use of tensile, compressive and shear (but not grip) stresses in concrete and steel up to 33⅓% in excess of compressive stresses of 750 lb. per square inch in 1 : 2 : 4 concrete and 1000 lb. per square inch in 1 : 1 : 2 concrete and a tensile stress of 18,000 lb. per square inch in steel reinforcement provided that such excess is due solely to wind pressure and that the district surveyor be satisfied under regulation No. 3 that the building has a margin of safety of two to one.

Workmen's Flats, Stepney.

An interesting example of reinforced concrete applied to the construction of working class flats (Fig. 1) has been completed during the past year. The architect for the building, which is in Lower Chapman Street, Stepney, E., is Mr. Joseph Emberton, F.R.I.B.A., and the reinforced concrete work was designed by The British Reinforced Concrete Engineering Co., Ltd., who also supplied the reinforcements; the contractors are Messrs. E. R. Hipperson & Son.

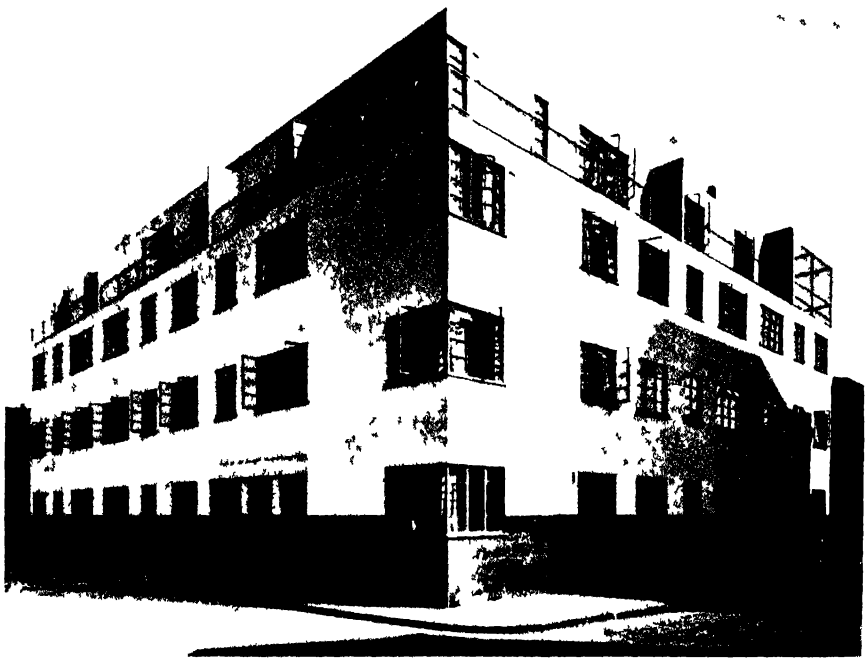


Fig. 1.

The structure, comprising a ground floor, three upper floors and a flat roof, extends along two sides of the site, the remaining area forming a courtyard from which an external staircase leads to the balconies giving access to the flats on upper floors. A portion of the flat roof at third floor level is used as a children's playground and the flats at this level have verandahs overlooking the street. The two arms of the building are separated by an expansion joint, and an open stair structure is independent of the main building.

The foundations consist of mass concrete piers tied together with reinforced beams and are carried down to gravel, a satisfactory stratum being found at varying depths. The ground floor is a panelled concrete slab laid directly on prepared filling and is separated by dry joints from the walls. The upper floor and roof slabs are of hollow tile construction with composition finishes overlying the concrete of the floors and with asphalt on the roofs.

With the exception of the partition walls between the apartments in the flats, the internal and external walls are of reinforced concrete. Columns at the intersection of longitudinal and transverse walls and floor beams are formed within the wall thickness.

The inside faces of all external walls are lined with insulation boarding which was placed in the forms before concreting. The partition walls are in breeze blocks and all internal surfaces are rendered. The external surfaces of the walls are treated with cream Cintocrete with a black dado at ground level.

New Methods of Construction.

NEW TYPES OF COLUMNS AND FLOORS.

In an extension to Shropshire House, Pancras Street, W.1, and the construction of Neville Court, an eight story block of flats, two new methods of construction were adopted by the consulting engineer, Mr. Frederick Rings

New Type of Column.

At Shropshire House (*Fig* 1) the design presented considerable difficulty owing to the fact that there is a roadway entrance on one side and the adjoining owner would not allow more than three columns to be placed against his building; the projection of these columns was also restricted to 9 in About one-half the weight of the building had to be carried on these three columns, as the architect wished to avoid the use of internal columns. As the live load of 1 cwt. per foot super could not be reduced, the weight of the structure had to be maintained at a minimum It was found that a reinforced concrete structure with solid floor panels would weigh 1,200 tons, but, in order to reduce the three roadway columns to the required size of 9 in., the weight of the building could not exceed 1,000 tons. The difficulty was solved by adopting a new type of floor (described later) which saved about 100 cb yd of concrete and a considerable amount of steel

It was clear that the three 15-ft. columns would have been too bulky if constructed in ordinary reinforced concrete or in structural steel The difficulty was overcome by using a new type of column (*Fig* 2), known as the "Multiple" column, which is designed to reduce the bulk of reinforced concrete columns and to allow continuity of the beam bars (*Fig* 3) over the supports, an impossibility in the case of steel stanchions owing to the presence of the web A "Multiple" column consists of four or more solid steel shafts, arranged symmetrically about two axes at right angles, the ends of which are let into solid steel caps and bases, the latter being bolted together in the usual way. The whole of the compressive and tensile stresses due to bending is resisted by the steel.

The column is designed so that the whole axial load is taken by steel, instead of by means of a combination of steel and concrete, thus combining in a practical manner the advantages of the structural steel stanchion with those of

Fig. 1.

the reinforced concrete beam. Each shaft takes its proportion of the full equivalent load, which is an assumed axial load equivalent in effect to the actual load plus an allowance for eccentricity and for bending due to beams, wind-pressure, etc.

If L is the length in inches of a four-shaft column, G its gyration radius, and R represents the ratio $\frac{L}{G}$ and A_s is the cross sectional area of one shaft, then where I_s is the moment of inertia of a single shaft, the minimum spacing D of the shafts for a given value of R is obtained by equating these two values of I_c. In practice the distance D is usually predetermined within small margins by the clear space required for the passage of beam reinforcements.

The radius of gyration of each shaft being less than that of the column as a whole, the permissible free length of shaft (l) is less than L, and cross-ties

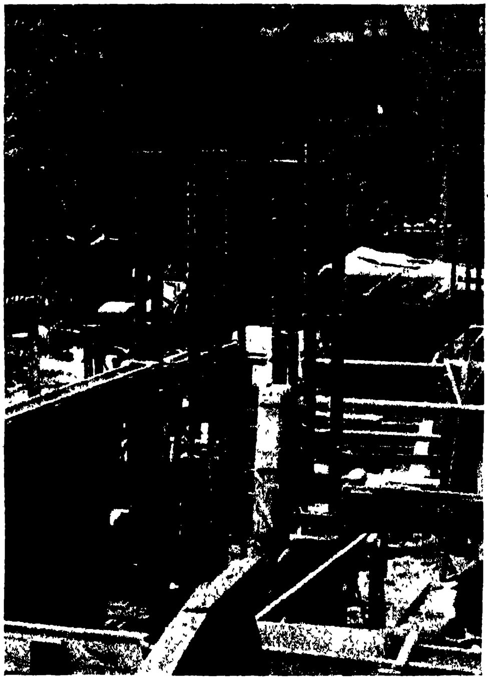


Fig. 2.

the permissible stress f must not exceed the value corresponding to R tabulated in the L.C.C. Steel Code, Section 40*b*. In calculating G and the moment of inertia of the column the concrete is neglected.

The minimum permissible moment of inertia of a column for a given value of R is thus

$$I_c = \frac{4.A_s.L^2}{R^2}$$

and since the actual moment of inertia of the column is

$$I_o = 4\left(I_s + A_s.\frac{D^2}{4}\right) = 4.I_s + A_s.D^2$$

are introduced so that the ratio $\frac{l}{g}$ is not greater than R, where g is the gyration radius of the single shaft.

The shafts are enclosed in spiral hooping which acts as a web, and resists shear stresses due to buckling or external bending forces.

The ends of the column shafts are reduced to one-half the diameter of the shaft, driven through the baseplates and caps and electrically welded on the far side, the faces of the plates being finally machined. The plates, are square or oblong in shape and are large enough

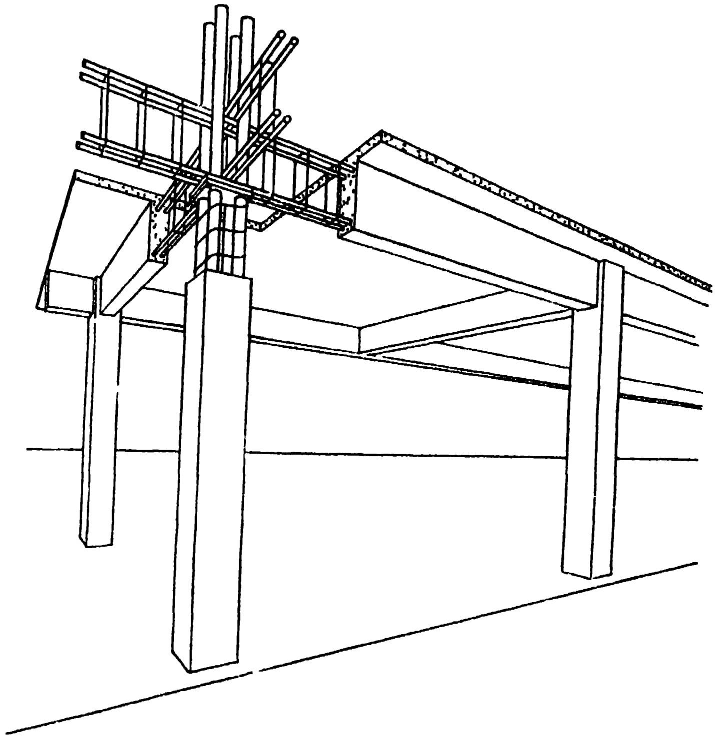


Fig. 3.

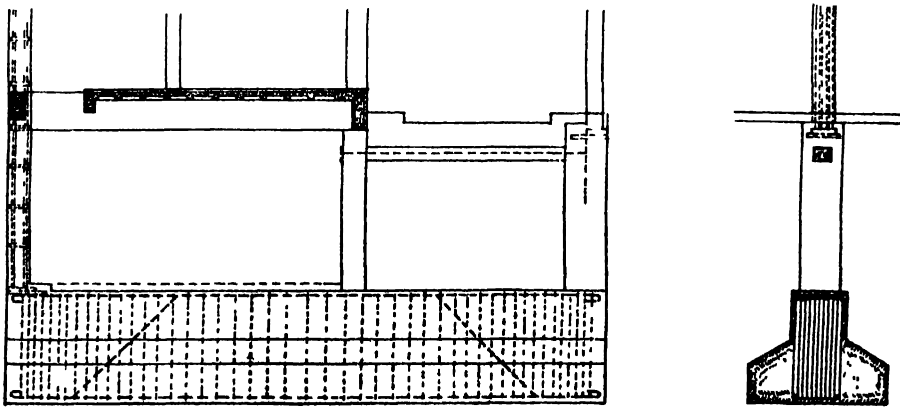


Fig. 4.

to accommodate the column shafts and anchor bolts with due regard to the provisions of the L.C.C. Steel Code. They are provided with a central hole to facilitate the pouring and rodding of a concrete core.

Where the positions of the shafts in successive tiers of columns coincide, the thickness of both caps and bases is one-third the diameter of the shaft. Where the positions do not coincide special calculations are made to determine the thickness of the plates.

The calculations for Shropshire House showed that two of the ground-floor columns required four 3-in. shafts each and a third six 3-in. shafts. In order to obtain a finished size of 9 in. the cover was made 1 in. of 4 to 1 granite grit and cement concrete, and it was successfully argued with the London County Council that this cover is more fire resisting than several inches of 1 : 2 : 4 ballast concrete. The Council also agreed to pass the "Multiple" column, which is virtually a steel column, as a reinforced concrete column.

It is of interest to note the difference in size of the three available types of columns. For instance, if a column to support 800 tons is 12 ft long, a reinforced concrete column would be 38 in. square, a steel stanchion cased in concrete 28 in. square, and a "Multiple" column 21 in. square. In arriving at the 38 in. mentioned the calculation was based on the proposed new Code of Practice, the concrete and steel being stressed to a high degree.

For the heavier loads the "Multiple" column also shows a saving in cost. For loads less than about 80 tons it may

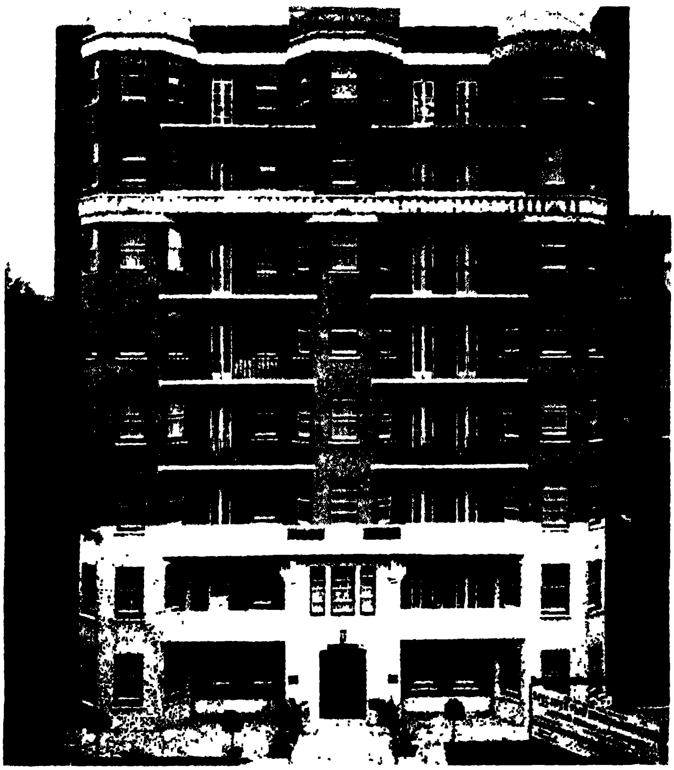


Fig. 5.

be argued that an ordinary reinforced concrete column is cheaper, but the difference is claimed to be counter balanced by the saving in labour, handling, and fixing

The whole of the foundations for Shropshire House were kept within the site (*Fig* 4) and a layer of felt was provided to separate the external wall panels from the walls of the adjoining buildings in order to avoid future complications

The architects for the building were Messrs Waite & Waite the consulting engineer Mr Frederick Rings and the contractors Messrs Sabey & Son (Islington) Ltd The floors were constructed by the Economic Floor Co Ltd and the Multiple columns by Messrs

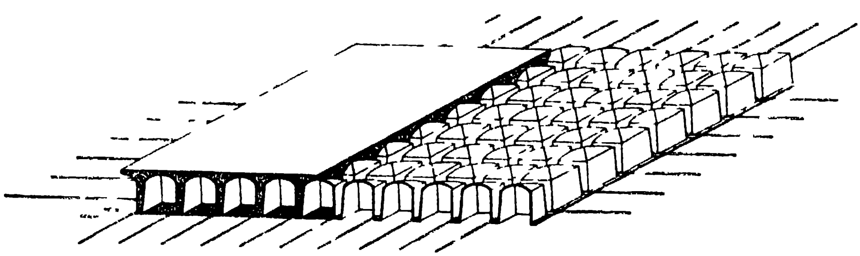


Fig. 6.

McCall & Co (Sheffield), Ltd who also supplied and fixed the whole of the reinforcement

New Type of Floor.

Neville Court (*Fig* 5) has a reinforced concrete frame and foundations The external walls are 14 in brickwork and the dividing walls and partitions are hollow the projecting bay windows being cantilevered out at each floor level The walls enclosing the staircase are in reinforced concrete so as to make cantilever stairs of the lightest type possible In order to reduce the size of the columns and ensure continuity of beam steel the ' Multiple column was adopted

A feature of the building is the use of a new type of floor, known as the ' Economic " floor (*Fig* 6), without a suspended ceiling This floor is monolithic and constructed by means of steel domes acting as temporary centering The domes have a 15-in square base, slightly tapering sides, and vaulted tops The rise or curvature of the top varies in proportion to the depth, and the minimum thickness of concrete on top of the domes is 2 in The floor is a grid in which both concrete and steel are fully stressed, but its main advantage is its lightness since a 6 in floor of this type weighs only 45 lb per square foot compared with 72 lb for a solid slab Thus a considerable saving is effected in the loadings of beams, columns and foundations

In this building the floors have a total depth of 4 in and the spans range up to about 14 ft When the domes forming the temporary centering are removed the soffits show a coffered or panel effect If a hollow floor is desired an expanded metal or fibrous plaster ceiling can be provided and fixed without difficulty Owing to the recesses in the floor sounds are diffused In this building the architect used the new floor without a suspended ceiling and it was found that, due to the steel centering, the soffits required very little after treatment to assure sufficient smoothness The soffits were painted with aluminium paint

The architect was Mr John F Yerbury, the consulting engineer Mr Frederick Rings, and the builders Messrs Griggs & Son, Ltd As in the case of Shropshire House the columns and reinforcement, of which there were about 170 tons, were supplied and fixed by Messrs McCall & Co, (Sheffield) Ltd

Piling in North London.

In the erection of a new store with flats above for Messrs John Barnes, of Finchley Road, for which Messrs T P Bennett & Son are the architects and Mr B L Hurst the engineer, the foundations presented a difficult problem owing to the fact that the Metropolitan Railway tunnel runs under the site and provision had to be made for a possible future tunnel running alongside the existing one The space available for the foundations carrying 22½ tons per lineal foot of wall was 2½ ft wide, so that piles taken below the bottom of the existing tunnel, and carrying 100 tons each, were decided upon

While piles could be driven on parts of the site, boring was resorted to in situations where the tunnel would have been endangered by driving The Franki Compressed Pile Co made tests by driving and loading a pile to make sure of its carrying capacity and by boring, by a new process provisionally patented by the company, alongside the tunnel In the driving test the pile stopped in medium soft clay at 30 ft, making it a semi-floating pile With a load of 160 tons, settlement was ¼ in

The Franki system of boring was then tried alongside the tunnel, where unexpected conditions were found Due to heat emanating from the tunnel or to the continuous vibration of passing trains the clay encountered, unlike that elsewhere on the site, was so dry and hard that it was impossible to drive through it In view of a possible new tunnel it was essential that the piles should go below the bottom level (about 30 ft) Tackles were rigged from the top of the driving tube to the bedframe of the machine and taken to the drum of the forward winch, so that a weight of approximately 35 tons could be taken on the tube, which was fitted with a special cutting edge In this manner the open tube was taken down 4 to 5 ft and then pulled up, bringing with it a cylindrical core of clay, which was expelled by a specially-devised hammer This operation was repeated until a hole 24 in (the outside diameter of the cutting edge) was made to a point below the underside of the tunnel The pile was then finished by the standard Franki system It was found that by this process vibration was reduced to a minimum at which no damage to the tunnel could result.

Segments of Clay Extracted by New Boring Process.

Hyperbolic Cooling Tower at Croydon.

RAPID CONSTRUCTION.

In 1930 a reinforced concrete cooling tower of hyperbolic shape was erected at the Croydon Electricity Station, and its erection was described in this journal for January, April and October, 1930 A second tower of the same type (Fig 1) has now been completed in connection with the enlargement of the electricity station which is being carried out to the designs of the Chief Engineer Mr I N Rendell Baker, A M I E E the designers for the two towers being Messrs L G Mouchel & Partners, Ltd the contractors for the second tower are Concrete Piling, Ltd

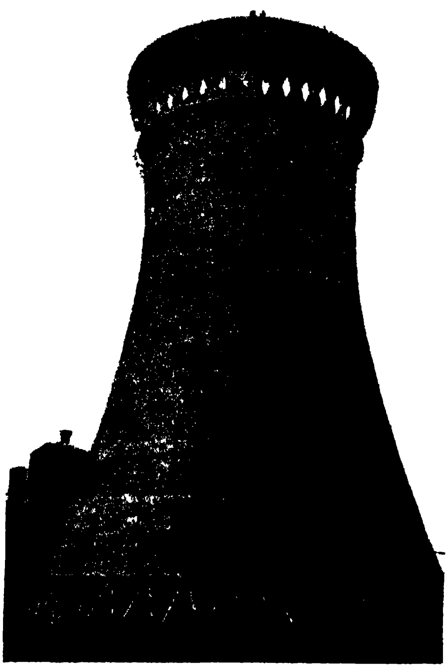


Fig 1

The obvious advantage of a reinforced concrete tower is the durability of concrete compared with wood, especially as the shell of a cooling tower is exposed to severe conditions conducive to the decay of timber Wood must still be considered for small towers, as the question of cost over a period of years must be balanced between the extra capital costs of concrete towers and the extra maintenance of wooden towers Mr Rendell-Baker has arrived at the conclusion that for a tower of over 100 ft in diameter and taking costs over a period of 30 years, the extra capital charges on a concrete tower of equal cooling capacity to that of a number of wooden towers are less than the charges for maintenance on the wooden towers and therefore the concrete tower is cheaper

Two other features in favour of the concrete tower are (1) owing to the greater height to which it is possible to build concrete towers, considerably better draught can be obtained than in the case of wooden towers which, for structural reasons, are limited economically to 110 ft (2) as the main shell of the tower acts as a chimney to induce air into the irrigation stack at the bottom it is essential that this should be air-tight This

condition is difficult to meet with wooden towers as the shrinkage of the timber sheeting after a few years causes openings of $\frac{1}{16}$ in or more between the boards these openings in the aggregate amount to a large area and cause a considerable diminution of draught within the tower by infiltration of outside air through the openings

Design.

The new tower has been designed to deal normally with 1,500 000 gallons of circulating water hourly The total height of the tower is 195 ft from ground level and the diameter 150 ft

The irrigation system consists of a number of large wooden troughs laid in radial directions and connected by smaller troughs laid circumferentially, from which the water drips and splashes on to sets of timber louvres laid at an angle of 45 deg with the horizontal Copper strips with copper rivets are used to join the various lengths of troughs The water is broken into a fine spray which evaporates quickly and its heat is transferred to the current of air rising in the shaft In order to cause as little obstruction as possible to the air current the sides of each trough are streamlined by sloping them inwards from the bottom to the top

The door in the shell which leads to the timber inspection gantry inside the tower is approached by concrete steps supported and reinforced by bars projecting from the outer face of the concrete shell In this way the usual steel staircase and landings and their maintenance are dispensed with

An interesting feature of the tower is the incorporation of a device for minimising the condensation precipitated from the vapour issuing when cooling towers are operating at maximum duty during periods of high humidity of the surrounding atmosphere This condensation is caused by the cooling effect of the outside air on the vapour-laden air issuing from the tower, which precipitates moisture due to the reduction in temperature The device consists of diamond-shape openings approximately 4 ft deep by 2 ft wide closely spaced around the tower at a distance of 20 ft from the top

Owing to the reduced pressure within the tower caused by the rising column of vapour-laden air, cool air from outside is induced through these openings and the mixing of this cool air with the warm vapour-laden air within the tower promotes precipitation of moisture within the tower shell Although the primary advantage of this device is to minimise

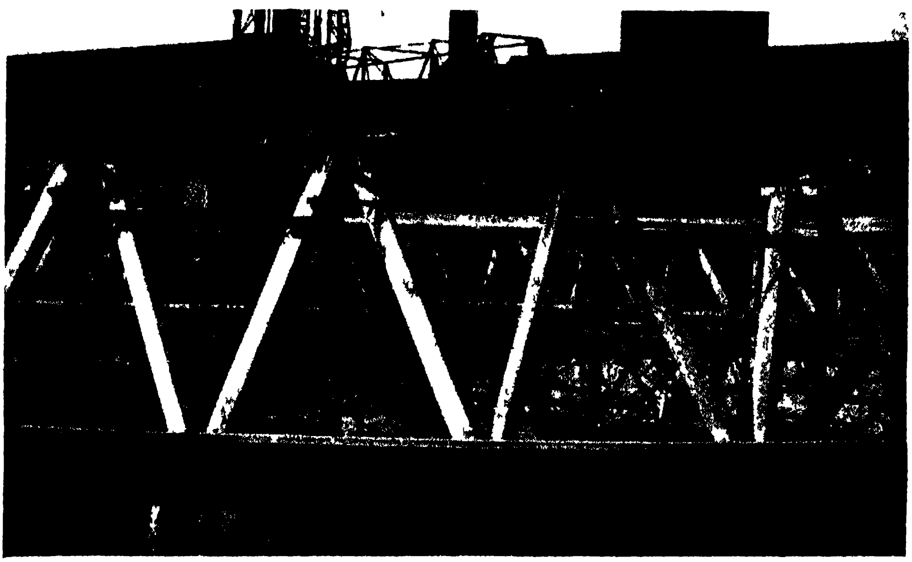


Fig. 2.

the nuisance caused by descending precipitated moisture on surrounding property, a further advantage is that as precipitation takes place within the tower shell the precipitated moisture descends within the tower, the water being thus saved for further use in the circulating water system. This interesting feature has been evolved by Mr. Rendell-Baker and his staff.

The cooling water is supplied from the Council's sewage farm at Beddington, and the shell is coated on the inside with bitumastic paint to prevent the growth of algæ from the effluent.

In the area occupied by the electricity works the subsoil is variable in character, but at the site of the cooling tower good ballast was encountered with only a small quantity of water in a few places.

Construction.

A feature of great interest on this contract was the original method adopted by the contractors to raise and place the concrete in the shell and to erect and strip the forms by a rotating bridge specially designed by the contractors for use in constructing this type of cooling tower. The bridge (*Fig. 2*) is so designed that it can pass through the top of the tower and also be extended to reach the sides of the tower below the top where the diameter is greater. The bridge may be raised as concreting proceeds up the tower

A circular jubilee track (*Fig. 3*) was laid on the floor of the pond from which materials could be raised to the bridge.

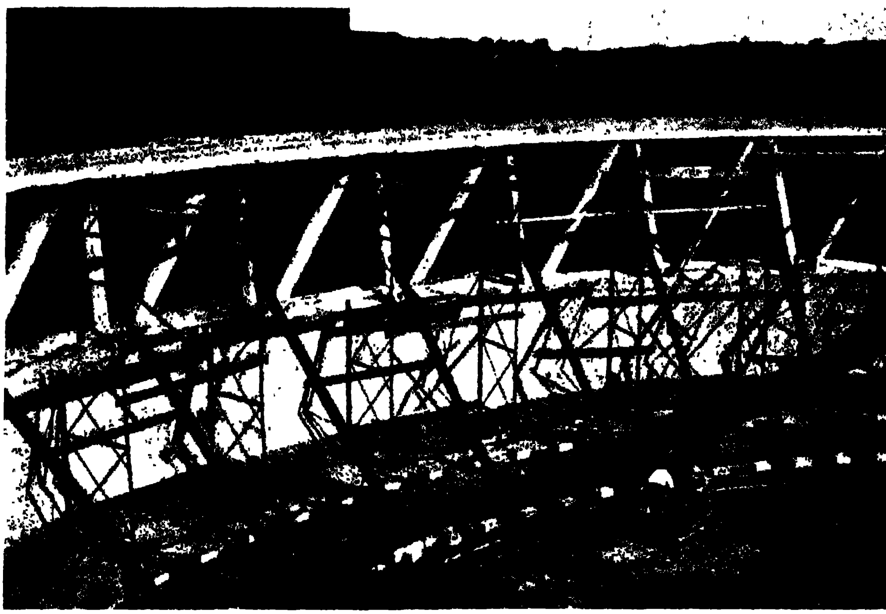


Fig. 3.

The only scaffolding required for the construction of the cooling tower was that from which the work below the upper intersections of the octagonal reinforced concrete bracing was built (*Fig. 3*). This consists of braced angle-iron stanchions in pairs with timber rakers and bottom sills All the remaining work was constructed from the rotating bridge, and the painting of the interior surface of the shell with bitumastic was executed from hanging cradles.

As an example of the speed at which the work was carried out the following is of special interest. When concreting the walls of the throat at a distance of

about 100 ft below the top of the tower, 11⅛ cb yd of concrete were raised and placed in the 4½-in wall in 4 hours The radius of the inner face of the wall at this height is 43 ft 9¼ in The rate at which the tower walls were concreted varied with its radius, and at the level mentioned was 12 ft in a week At the same level 185 sq yd of forms 3 ft high were erected in 9 hours by 4 carpenters

The existing ground level on the site of the tower being above the level of the floor of the pond an opening was left in the concrete wall of the pond (*Fig* 5) and the aggregates were dumped alongside this opening by lorries and descended by gravity to the loading hopper of the 10-7 cb ft concrete mixer inside the wall From the mixer the concrete was run in jubilee skips to the foot of the steel tower, where it was discharged into the skip travelling inside the tower. On arriving at the decking level of the bridge the concrete was transferred to barrows and wheeled to the forms

The shuttering used (*Fig* 5) was the Concrete Piling Company's pattern in which the panels are 4 ft 6 in long by 3 ft wide The framework of stiffeners consists of steel angle-irons and the sheeting is of timber which can be renewed after it becomes worn

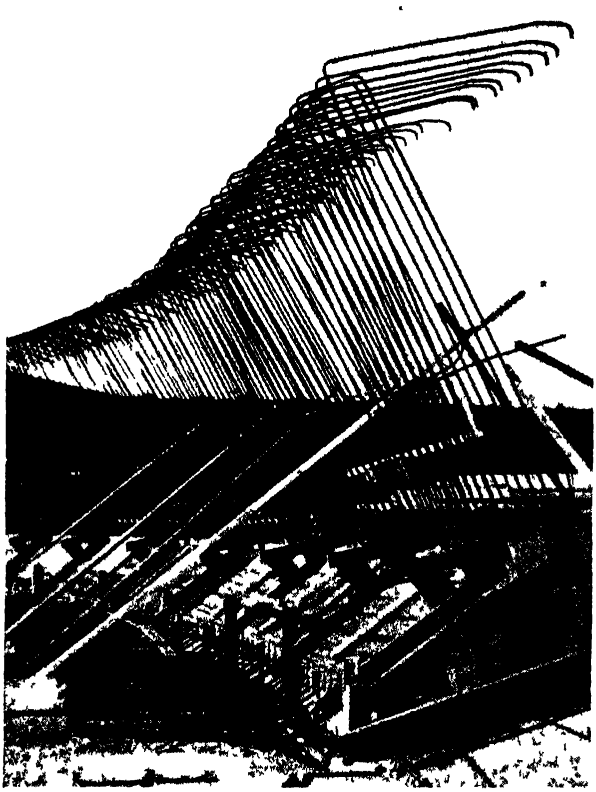


Fig. 4.—Reinforcement of Pond Wall.

The concrete for the reinforced work was mixed in the proportions of 7 cwt of ordinary Portland cement to 13½ cb ft of sand and 27 cb ft of crushed ballast The reinforcement was obtained from the Whitehead Iron & Steel Co , Ltd.

Plywood radial joints were made in the coping of the pond wall at the levels of the lower intersections of the diagonal bracing.

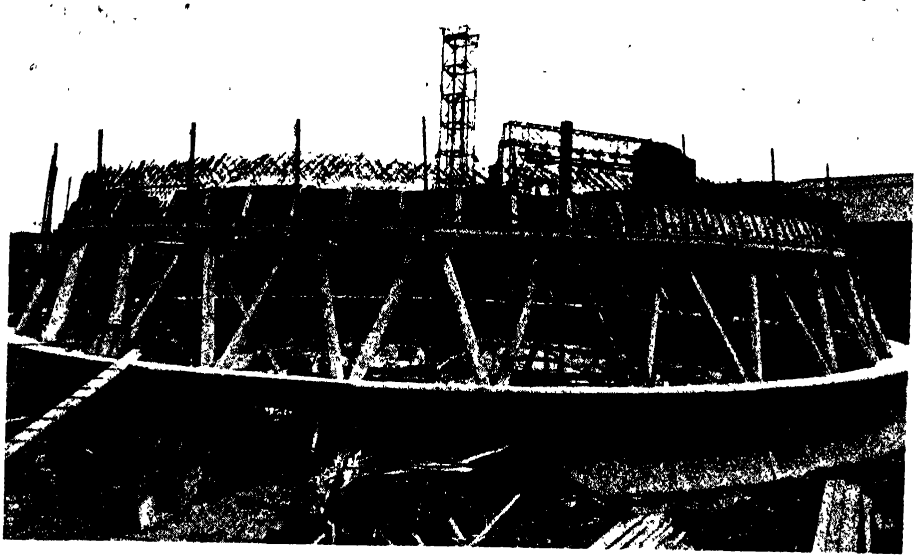


Fig. 5.

Culvert.

Between the cooling pond and the existing culvert at the generating station a new reinforced concrete culvert 800 ft. long has been constructed. This is rectangular in section, 10 ft. 6 in. high by 7 ft. wide internally, with 8-in walls and floor and a 6-in. roof slab Where the culvert passes under Factory Lane the section is 6 ft high by 7 ft. wide, with 12-in walls, floor, and roof No water-proofing medium was used in building the culvert

Kent Coastal Road, Faversham-Whitstable.

This concrete road has recently been constructed for the Kent C.C. It is 10 in. thick reinforced with two layers of B.R.C. fabric and was completed under the direction of Mr. H. T. Chapman, M.Inst.C.E., the County Surveyor.

Paper Mill Bridge, Little Baddow.

THE application of reinforced concrete screw piles for bridge foundations is exemplified in the following description of the construction of Paper Mill bridge, Little Baddow, which has been designed by Mr. R H Buckley, M Inst.M.&Cy.E, County Surveyor of Essex, and which is shown during construction in *Fig.* 1.

The bridge carries the Hatfield Peveril–Little Baddow road over the Chelmer and Blackwater canal and comprises a middle span of 25 ft. and two 15-ft. side spans carrying a 29-ft road and two 5-ft. footpaths. The superstructure consists of pressed steel troughing 2 ft 8 in by 12 in. by ½ in in section which will be filled with 1 : 2 : 4 concrete and surfaced with tar-macadam. Mass concrete retaining walls founded on gravel carry the outer ends of the 15-ft. spans, and the river piers are supported on "Screwcrete" reinforced concrete piles. The latter are arranged in three groups of two under each pier (*Fig.* 2), and the reinforcing bars in the piles are bonded into reinforced concrete spreader beams between the two piles of each group. Above these beams are A-frames carrying 14-in. by 5½-in. by 40-lb. rolled steel joists encased in concrete which distribute the loads supported by the troughing. The bridge crosses the river at an angle of 15 deg. 54 min.

As a protection to the piles in case of craft fouling the river piers, the latter are surrounded by precast concrete shells supported on reinforced concrete foundation slabs 8 ft. long by 3 ft. 9 in wide enclosing two piles and located below the river bed. The slabs were precast in 4-ft. by 3-ft. 9-in. sections and threaded over the piles, and the shells were filled with rough concrete when fixed. A reinforced concrete beam connects the tops of each pair of piles and is enclosed in a cast-in-situ concrete cap formed above the filling of the shell.

Fig. 1.

The shells are 7 ft. 6 in. long by 3 ft. wide by 2 ft. deep with semi-circular ends and a thickness of 5 in. Two cranked steel bars connect the opposite sides of each shell; these are used for the attachment of the crane slings when the shells are being lifted and placed in position.

Borings indicated that the underlying strata consisted of sand and mixtures of sand and clay or ballast to a depth of 30 ft., and for this reason the "Screwcrete" pile was adopted since it gives a large bearing area. The pile shafts are 14 in. in diameter and are fitted with a 3-ft. 6-in. diameter helix. *Fig.* 3 shows the 12-gauge steel casing and helix assembled and supported by the derrick. When the pile is pitched the steel mandrel

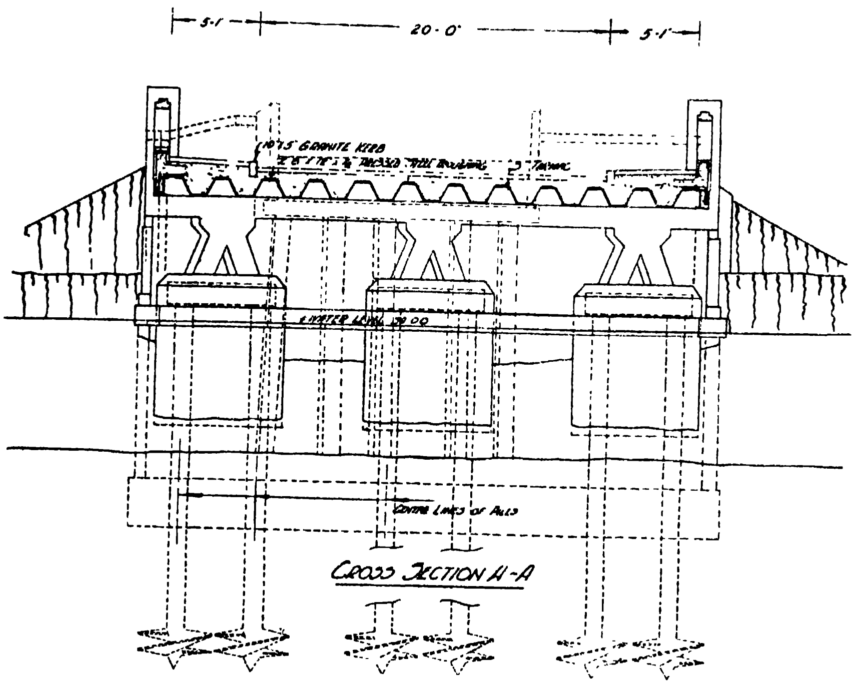


Fig. 2.

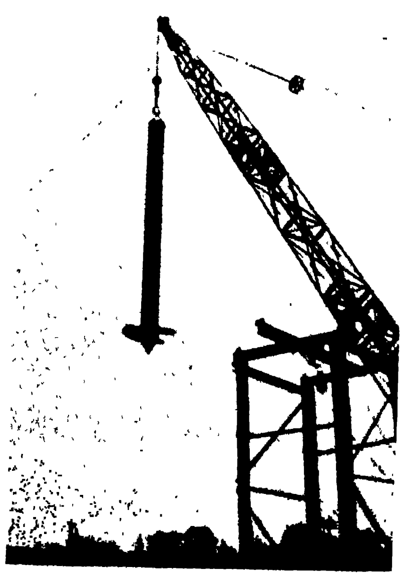


Fig. 3.

Fig. 4.

(*Fig* 4) is lowered inside the casing and connects with the helix The electrically driven screwing capstan is then placed on the mandrel and screwing is started The reaction to the torque of the capstan is taken by the rolled steel joist cage shown in the photograph, and this is in turn guyed back to suitable anchorages

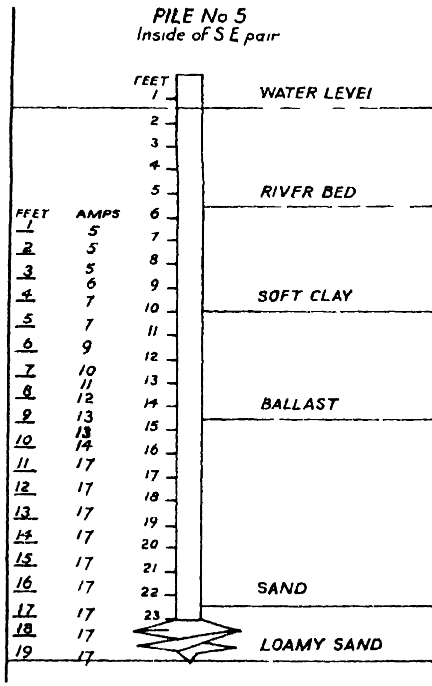


Fig. 5.

The time required for screwing a pile was two hours for a distance of 15 ft, and the deepest shoe was screwed 21 ft 9 in through the ballast and into the sand The diagram (*Fig* 5) is typical of those prepared on the site as a record of the resistance of the subsoil to the screwing operations The current consumption provides a reliable guide as to when the helix has reached a suitable level On completion of screwing a cage of six ⅞ in bars was lowered into each pile and the tube was filled with 1 2 4 concrete Construction of the upper portion of the piers followed Ordinary Portland cement was used

The contractor is Mr A G Wicks, of Braintree, and the sub-contractors for the piling and steelwork are Messrs Braithwaite & Co, Engineers, Ltd Work was commenced in June of last year and is expected to be completed in February 1935 No interference with the barge traffic has occurred during the construction

Royal Docks Approaches.

SILVERTOWN DIVERSION.

IN May 1933 and January 1934 descriptions were published in CONCRETE & CONSTRUCTIONAL ENGINEERING of the works at East India Dock Road including the new bridge over the river Lee and the ¾ mile reinforced concrete viaduct between Barking Road and North Woolwich Road which form part of the scheme of improved approaches to the construction of the Silvertown diversion which is now in its final stage.

At Silvertown station a diversion has been made by a viaduct which eliminates the level crossing at that point. The contractors were Messrs. Holloway Bros. (London) Ltd. The new road is carried over the lines of the London & North Eastern Railway Company and the Port

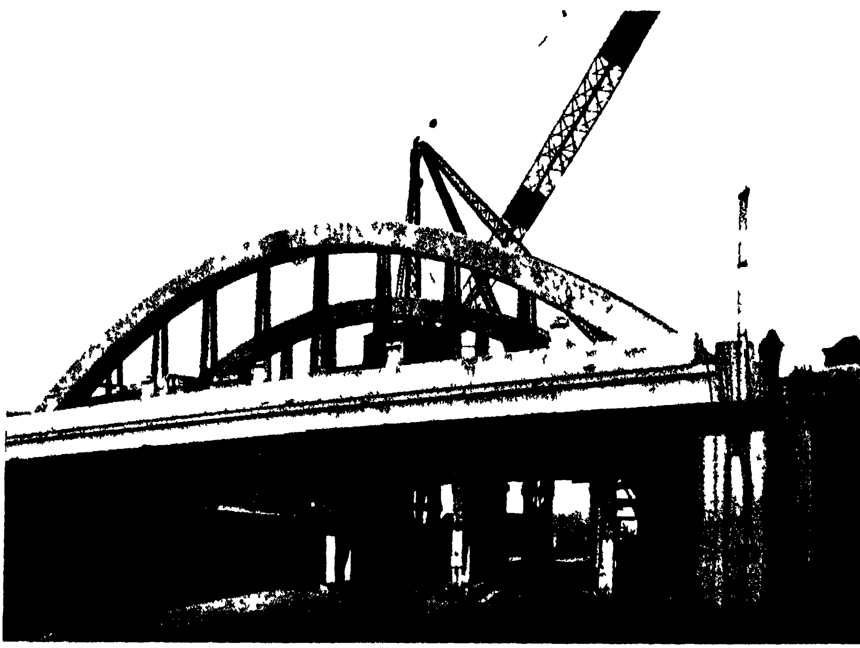


Fig. 1.

Royal Victoria, Royal Albert and King George V Docks.

In 1929 powers were granted to the London County Council and the County Borough of West Ham to construct new works and improvements which would reduce traffic blocks and make the flow of traffic more uniform and continuous. All the south-eastern counties, the City Corporation, and several county boroughs have contributed to the cost. Messrs. Rendel, Palmer, and Tritton are the engineers for the scheme. The improvement scheme will be completed by the of London Authority by a reinforced concrete viaduct connecting North Woolwich Road with Connaught Road, and having a length of about 580 yd.

The approach portions of the viaduct on each side of the central bridge over the railways have gradients of 1 in 30. With the exception of short lengths at both ends where the road slab is supported on solid filling, the construction may be divided into (1) standard unit construction, and (2) slab-and-wall construction. The slab-and-wall construction in each approach is 186 ft. long and

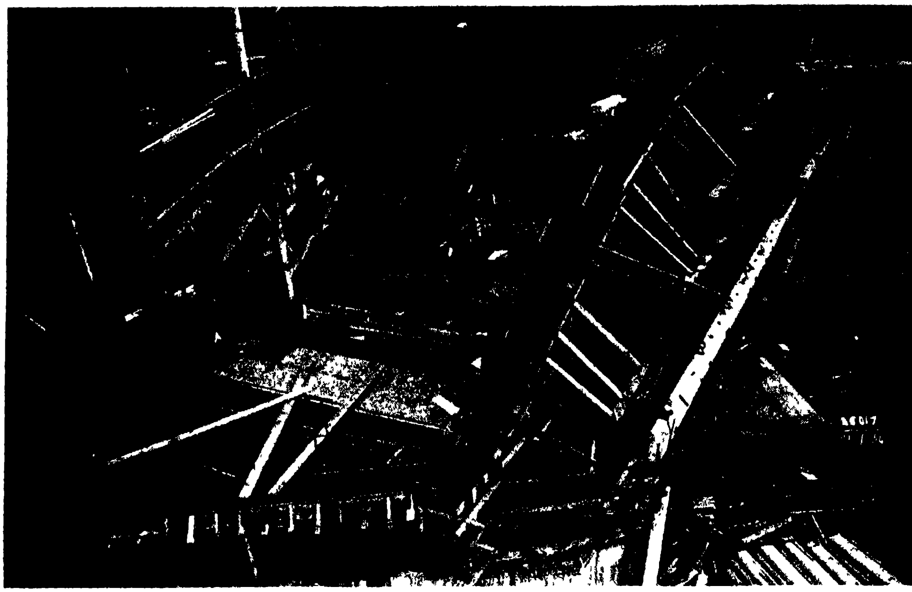


Fig. 2.

is substituted for the standard unit beyond the point where the falling gradient of the road reduces the height of the columns to about 12 ft. Over this section the road slab is 18 in. thick and is supported on cross walls founded on a single row of Vibro piles.

The standard unit consists of a section of the viaduct 62 ft. in length separated by expansion joints from the adjoining units. The reinforced concrete deck slab is 10½ in thick and is carried by beams and columns which stand on pile caps covering clusters of two, three, or four piles according to the load carried. The unit is divided into three bays, in which the columns are 22 in. by 22 in. and 22 in. by 19½ in. and spaced at 20-ft. centres longitudinally and 15 ft. transversely ; their maximum height is about 23 ft. The pile caps are connected by beams transversely. In all there are 900 17-in. Vibro piles in the contract.

A bowstring girder bridge of reinforced concrete (*Fig.* 1) with a skew span of 109 ft. clear width carries the road over the London & North Eastern Railway and Connaught Road. There are, in addition, four reinforced concrete rigid frame bridges, with spans from 40 ft. to 43 ft. A spur road from the high level will give access to the Royal Albert and King George V Docks.

The bowstring girder bridge carries a 28-ft. roadway between the ribs, which are 20 ft. high above road level at mid-span and are spaced at 34-ft. 6-in. centres. A view of the bridge and the formwork for the ribs is shown in *Fig.* 2. Outside the ribs are two footpaths cantilevered off the cross girders.

Graphs for Design of Reinforced Concrete.

A USEFUL series of curves for determining sections to resist given forces and the stresses in trial sections is contained in " Tables de Calcul Direct " (Paris : J. M Peigues, 56 rue N.D. des Champs. Price 20 fr.) compiled by M. G. Ledeut. These are based on the fundamental equation $f = \frac{My}{I}$, in which the moment of inertia of the equivalent section is used. A number of examples is included to illustrate the use of the charts. The diagrams are based on a modular ratio $m = 15$.

Leckwith Bridge and Viaduct, Cardiff.

WORK is nearing completion on the new, bridge and viaduct (*Fig* 1) at Leckwith, near Cardiff The scheme includes a reinforced concrete arch bridge over the river Ely a high level reinforced concrete viaduct 320 ft long and approach works making a total length of approximately 600 yd and has been designed to by pass an existing narrow bridge and dangerous corner and at the same time reduce the present gradient from 1 in 8 to 1 in 15 With the exception of the northern approach the work is being carried out by the Norwest Construction Co Ltd for the Glamorgan County Council and the Cardiff Corporation the river Ely being

Fig. 1.

a boundary between the areas of these authorities

The bridge has a single span of 100 ft on a skew of 54 deg and is an open spandril arch rib type, carried on mass concrete abutments The four ribs are 4 ft 6 in deep by 5 ft 6 in wide at the springing, tapering to 3 ft 4 in deep by 4 ft wide at the crown, and the deck is a beam and slab design carried on reinforced columns at 10-ft centres Tie beams are provided between the arches at the quarter points and at the springings The mass concrete abutments contain approximately 1500 cb yd of concrete each, foundation level being at – 4 00 O D. which is 14 ft below the river bed level or 42 ft below deck level

A 30 ft carriageway is provided and two 6 ft 6 in footpaths The parapets are of reconstructed Portland stone uniform with those of the viaduct The bridge designs were prepared by Mr T H Morris under the direction of Mr G H Whitaker M Inst C E the City Engineer and Surveyor

The viaduct is a continuous beam and slab construction with eight 40 ft spans and a triangular end span adjacent to the skew bridge It is on a 1 in 15 gradient and on a 613 ft radius in plan the main beams are straight between the columns but the reconstructed stone parapets are set to the radius The floor is superelevated the amount of super elevation giving sufficient depth for a pipe duct on the outer curve of the viaduct The columns are tapered in elevation and the four columns of each trestle are cross braced as shown in *Fig* 2

For the foundation of each column four Vibro piles were driven at each column position to depths varying from 18 ft to 30 ft the specified set was ½ in for ten blows of a 2-ton hammer falling 4 ft This piling was carried out by the B S P licensees The pile heads are connected by a grillage as shown in the illustration, and form part of the column bases shown in *Fig* 2 The column bases are connected below ground level by longitudinal tie beams which are

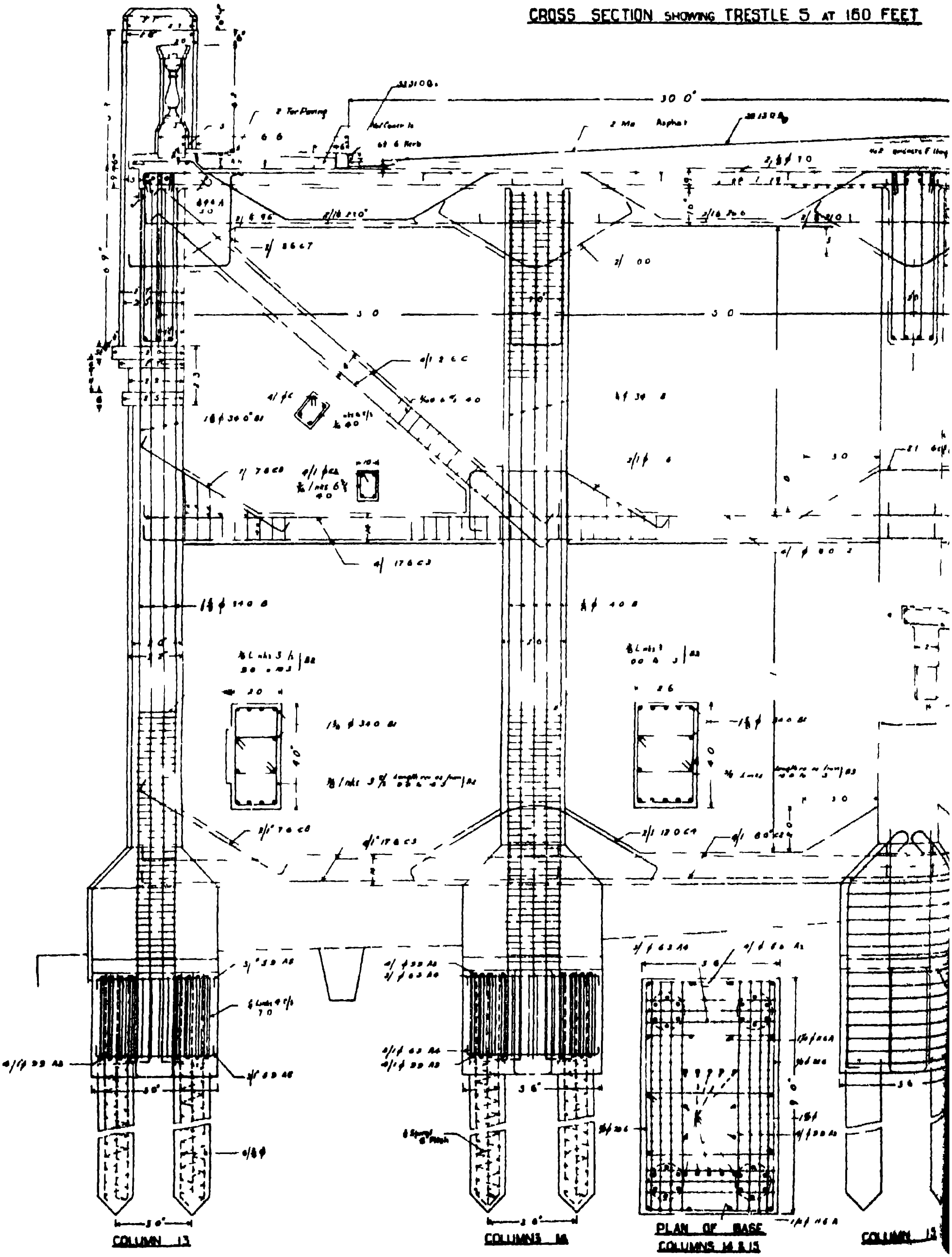
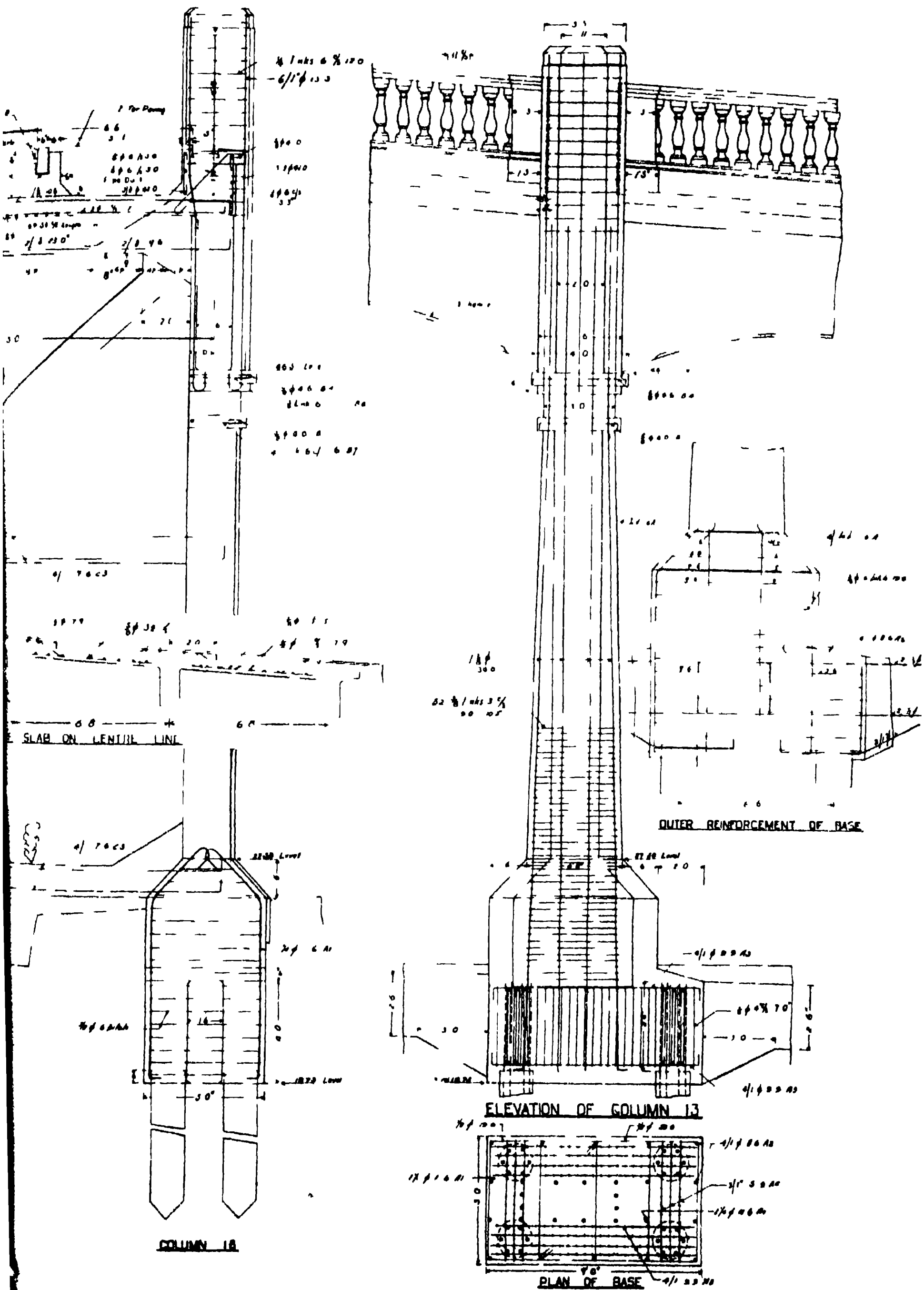

CROSS SECTION SHOWING TRESTLE 5 AT 160 FEET
COLUMN 13
COLUMNS 14
PLAN OF BASE
COLUMNS 14 & 15
COLUMN 15

Fig. 2—Le


SLAB ON
COLUMN 18
ELEVATION OF COLUMN 13
OUTER REINFORCEMENT OF BASE
PLAN OF BASE
COLUMNS 13 & 18

and Viaduct.

carried throughout the structure to the back of the mass concrete abutment of the bridge At the southern end of the viaduct the ground rises sharply and the underlying rock lies within 10 ft to 14 ft of the surface Prestcore piles were used for the column foundations in this position

The mass concrete abutment at the southern extremity of the viaduct has a base thickness of 17 ft at 9 ft below average ground level the thickness being reduced by 12 in steps to 5 ft 3 in at the main beam bearing level This abutment with its wing walls also acts as a retaining wall for the heavy embankment approach, the depth of filling being 18 ft at this point Expansion joints were provided at the extreme ends of the structure, and the main beams at the free ends are supported on cast steel bearing plates with copper alloy plates as sliding surfaces

The contracts for the bridge and viaduct were let to the Norwest Construction Co, Ltd, and work was commenced in July 1933 Notwithstanding the tidal nature of the river, with water level fluctuating from 11 0 to 25 0 O D, no more than ordinary difficulties were experienced in building the cofferdams (*Fig* 3) for the abutments Larssen No 2 section piles were used in 37-ft lengths, and a sluice pile was provided to deal with differences of water level in case of emergency As the springing of the arches was below mean tide level it was necessary to build out special coffer boxes from the face of the cofferdam, one to enclose each arch rib, within which the shuttering could be aligned and the arch rib steel fixed free from interference and river silt These coffers were made of 3 in tongue and groove timbers supported on the bridge centering and fixed to the face of the cofferdam piles after which the piling on the face enclosed was burnt through and removed Some difficulty was experienced in the first abutment due to the rise and fall of tide causing the joints between arch rib coffers and steel piling to "breathe" slightly, but this defect was remedied by a slight alteration in design and did not occur on the second abutment

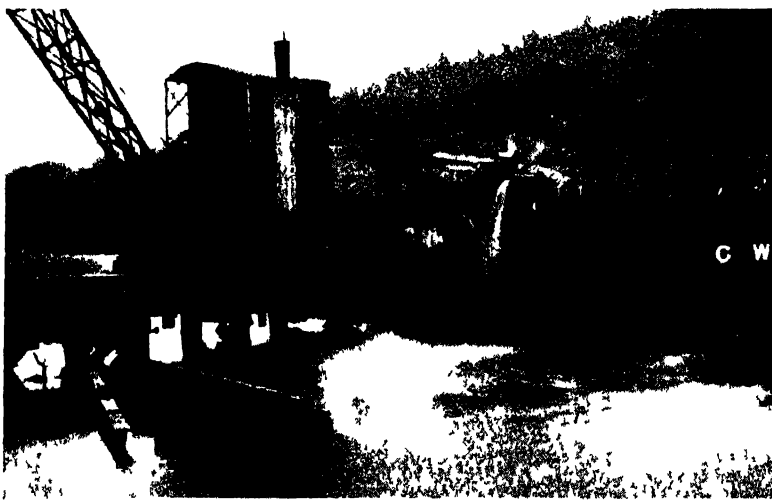


Fig. 3.

As two shrinkage keys were provided in each arch rib at about the quarter points, concreting was arranged to proceed slowly but continuously to these points without a stop, and the same procedure was employed in casting the centre portion of each rib between the shrinkage keys This method avoided unnecessary concrete joints, which would have been difficult to hack in view of the size and number of the reinforcing bars

The falsework of the arch ribs and

superstructure was composed of 20-in. by 6½-in. rolled steel joists supported on 12-in. by 12-in. timber piles, the greatest clear span of the joists being 25 ft. An additional rise of 1¼ in. was allowed in setting the arch rib shutters to cover deflection in concreting and stripping.

The concrete mixes specified in the bridge contract were: 22 cb. ft of crushed graded stone, 11 cb ft. of Channel sand, and 3½ cwt of Portland cement for mass concrete, and 22 cb. ft., 11 cb. ft., and 6 cwt. of Portland cement for reinforced concrete. Special care was taken in grading the aggregate to obtain a fineness modulus of 5·50, and allowance was made for bulking of the sand. A uniform and satisfactory concrete resulted from the attention given to these details.

The foregoing remarks apply also to the concrete used in the viaduct, except that for the reinforced work rapid-hardening Portland cement was used in place of ordinary Portland cement

In the construction of the viaduct the main difficulties encountered were due to the sidelong and headlong nature of the site The centering was supported mainly from the longitudinal tie beams previously mentioned. In designing the centering for the bridge and viaduct the stresses allowed in the timber were those recommended in Mr. A. E Wynn's handbook " Design of Formwork for Concrete Structures," as these have been found satisfactory in the past.

An unusual difficulty was met during excavation for the south approach, where a considerable section of the hillside had to be excavated to accommodate the new line of road. This excavation revealed a natural fault plane in the ground a little below the level of the old road, and immediately the cut was made a large section of earth measuring 200 ft. in length, 12 ft. thick, and apparently 150 ft. wide commenced to move forward along this plane. The amount of movement measured on the face was ¼ in. on the first day, and before the slip was finally checked the total forward movement was approximately 9 in. at the worst point. Heavy timbering was immediately adopted as a temporary expedient until proper investigation could be made and the retaining wall intended for this position redesigned by the County Engineer's department. It is estimated that the weight of earth in movement at one time was about 20,000 tons along a plane inclined at 15 deg. to the horizontal, this plane being formed by a thin layer of vegetable matter on a bed of greasy clay. Automatic electric signals were employed to control traffic during the reconstruction of the approaches, and functioned very satisfactorily in a difficult situation.

The design of the viaduct and south approach works was prepared by Mr. E. J Powell, A.M.Inst.C E., of the Glamorgan County Bridge Department under the control of Mr. E C Pole, the County Engineer and Surveyor, to whom and to Mr. G. H Whitaker, M Inst.C E., City Engineer of Cardiff, we are indebted for permission to publish this article. We are also indebted to Mr. W Williams and Mr W. Russell Morgan, the resident engineer, for their assistance and co-operation

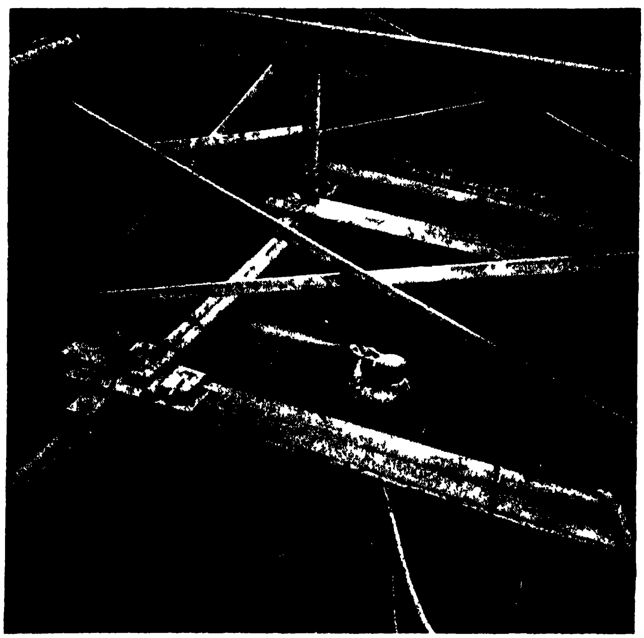
A Large Aeroplane Hangar.

THE area covered by the French Air Ministry's hangar completed at Berre in 1934 is 183,000 sq. ft. The main building is 670 ft. wide by 230 ft. deep and is divided by fireproof walls into three units 220 ft. by 230 ft., 230 ft. by 230 ft., and 220 ft. by 230 ft. in plan. The clear height of opening at the doors is 40 ft., and there are no interior columns between the walls. Between the four main reinforced concrete portal frames which span the three units the roof is covered with reinforced concrete arches. The main frames have spans of 220 ft., 230 ft., and 220 ft. and were designed as box girders with a depth of about 15 ft. The maximum moment in one frame due to the design load is about 28,000 ft.-tons.

Gunite Bunker Linings.

THE new Tir John Power Station at Swansea is equipped with two raw coal bunkers with a capacity of 700 tons each and four pulverized fuel bunkers with a capacity of 100 tons each and a brief description of the work of lining them with 2 in. of reinforced gunite may be of interest as being typical of bunker lining work.

The cement gun plant was installed at basement level so far as possible directly beneath the bunkers. The top of each bunker is about 100 ft. above basement level. The application of the lining may be divided into three stages: first, fixing a mesh fabric of $\frac{3}{16}$ in. diameter rods at 3 in. centres in both directions to the angle cleats of the bunker plates by tying with wire with 3 in. laps wired at intervals of 18 in.; second, the application of the first coat of gunite approximately $1\frac{5}{8}$ in. thick incorporating the reinforcement and brought to an even surface by a light screeding; and finally the second gunite coat bringing the lining to a finished thickness of 2 in. and left without trowelling or screeding. Where horizontal projections occur a fillet was shot so that the lining forms a slope of 45 deg. to prevent the lodgment of coal.



An important factor in bunker work is the removal of the material which rebounds from the surface as the gunite is placed, as an accumulation of this in the hoppers must be avoided. This was dealt with by steel chutes fixed to the hopper mouths through which the rebound material was taken to the basement where any of it which was suitable was dried and re-used in further mixings. Communication by signal between the operators inside the bunkers and the men at the plant was maintained by electric bells.

The time for execution of the work, which was carried out by one gun, was nine weeks; in cases of urgency the speed of the work can be expedited by increasing the number of guns employed. The work was carried out under sub-contract to Messrs. L. Turner & Sons and under the supervision of the engineers, Messrs. S. H. White & Co., by The Concrete Proofing Co., Ltd., who are at present placing a 3 in. reinforced gunite lining to the bunkers at the new Fulham Power Station as sub-contractors to Messrs. Higgs & Hill, Ltd.

Design of Piled Jetties and Piers.

By OVE ARUP.

(*Continued.*)

IN the May 1934 issue of CONCRETE & CONSTRUCTIONAL ENGINEERING the writer discussed the arrangement of the piles in a jetty so as to obtain the maximum resistance against horizontal forces. In this connection the reactions produced in the various piles by such forces were studied, but so far the effect of any vertical loads which the jetty might simultaneously have to support has not been considered. If we take these into consideration it will, however, not affect the general result obtained, namely, that the piles ought to be arranged in raking trestles so as to meet in pairs at deck level, but it will affect the position and number of raking trestles required.

It is clear that, if the piles are already loaded to capacity, even a small blow will encroach upon the factor of safety; on the other hand, if it is the danger of uprooting the piles which limits the strength of the jetty against horizontal blows, then the safety of the structure will be increased by the addition of vertical loads. In other words, for a given position of the piles in a jetty there is one "ideal" vertical load—dead or live load—which gives the jetty its maximum power of resistance against horizontal blows. The power of resistance will be diminished when the load is increased as well as when it is decreased.

It is naturally of interest to determine this "ideal" load, as it may then be possible for the designer to arrange the dead weight of the jetty so as to get the maximum resistance from the piles. The variations in live load make it impossible to obtain the "ideal" load under all conditions, and it does not always pay to increase the dead load to the "ideal" figure. It may prove more economical to obtain the desired strength by increasing the number of piles, although in this case not all of them will be stressed to capacity. The aim of the designer should be, if possible, to make the dead load plus, say, one-half the average live load correspond to the "ideal" load.

This "ideal" load can be found if we ascertain or assume the ratio between the safe carrying capacity of the piles and the resistance of the piles against uprooting. The greatest capacity for the absorption of shocks is obtained when all piles are stressed to their maximum capacity simultaneously, that is, all compression piles to the maximum safe figure for compression and all tension piles to the maximum safe figure for tension or uprooting. The "ideal" loading is that which satisfies this condition as nearly as possible; only in exceptionally simple cases can it be fully satisfied.

Let us, as before, consider the case of four piles arranged symmetrically about a vertical axis. *Fig.* 21 is a diagram of the pile axes similar to the diagrams in *Figs.* 16 to 19. The axes of the four piles, Nos. 1, 2, 3, and 4, intersect in p, p', q, q', u and u'. The lines I and II are the main axes of the system, intersecting in the O point, and the lines aa' and bb' are the major and minor axes of the central ellipse.

Assuming—as a simple case—that the piles can take twice as much load in compression as in tension, say, 50 tons in compression and 25 tons in tension, we must try to find a force which as nearly as possible will produce a compression in piles Nos. 1 and 3 which is numerically twice the tension produced in Nos.

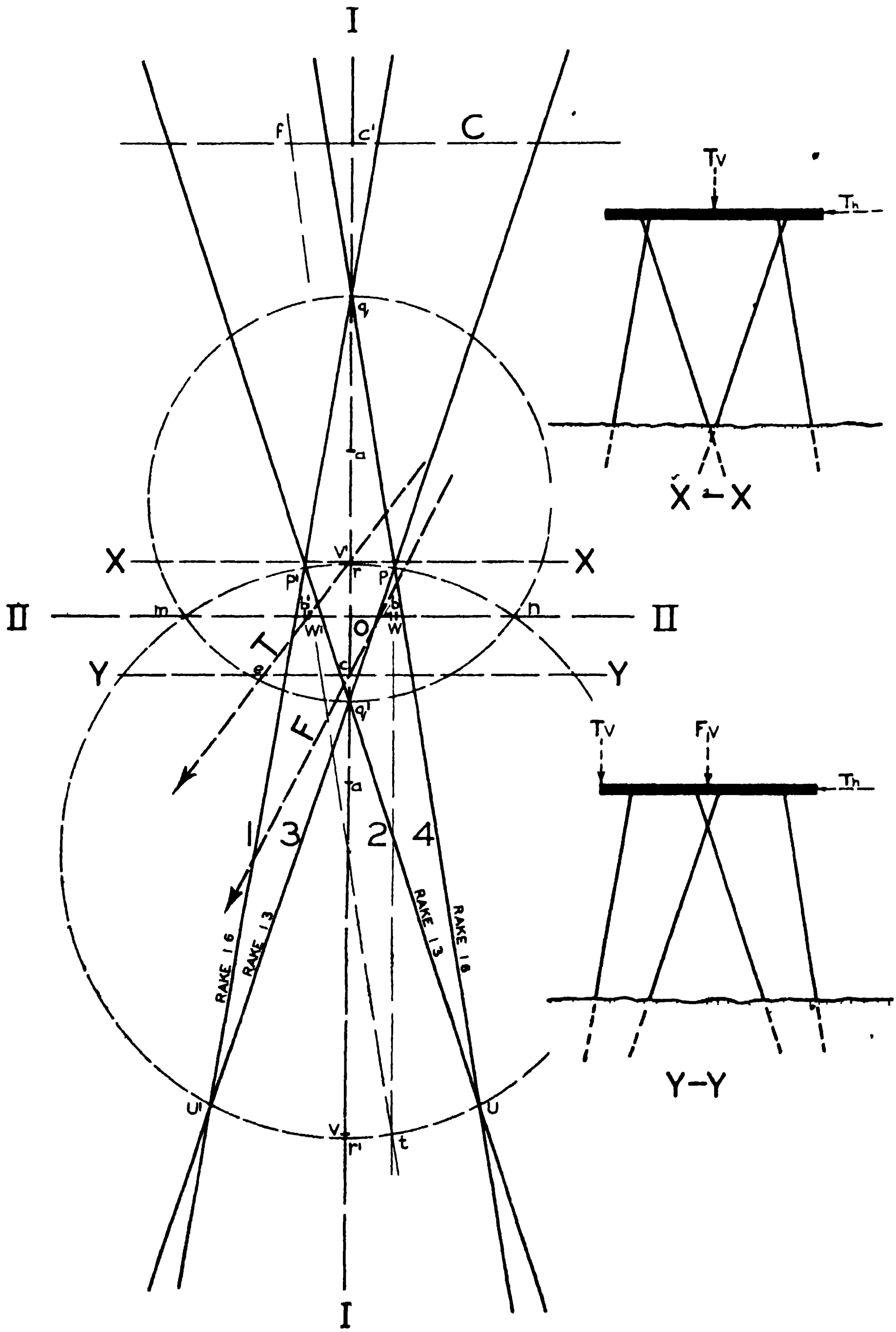

I
C
f
c'
Tv
Th
q
a
X
X
p'
p
r
II
II
m
n
b'
b
W'
O
W
Y
Y
e
c
T
F
q'
X–X
Tv
Fv
Th
1
3
2
4
RAKE 1:6
RAKE 1:3
RAKE 1:3
RAKE 1:8
U'
U
V
r'
t
Y–Y
I

Fig. 21.

2 and 4. It is known that the reaction produced in a pile by a given force is proportionate to the distance between the corresponding instantaneous centre of rotation and the pile axis. Therefore, if it is required to find the force to produce a compression in pile No. 1, which numerically is twice the pull in pile No. 2, the centre of rotation should lie on a line dividing the angle between 1 and 2 so that the distance from any point on this line to line 1 is twice the distance of this point to line 2. Similarly, if pile No. 3 is to have twice the load of pile No. 4, the centre of rotation must lie on the locus of points having a distance from 3 which is twice their distance from 4. These two loci intersect in t (on the circle p, p', u, u'). To satisfy these two conditions the instantaneous centre of rotation should therefore be t. However, it is also necessary to satisfy the two conditions that the reaction in 1 should be equal to the reaction in 3, and the reaction in 2 equal to that in 4. To satisfy the first, the centre of rotation should lie on a line bisecting the angle between 1 and 3 (line u' r') ; to satisfy the second it should lie on a line bisecting the angle between 2 and 4 (line u r'). These two lines intersect in r' (also on the circle p p' u u'). The instantaneous centre of rotation should lie on four different lines, p' t, p t, u' r' and u r' ; which is impossible unless the four lines meet in the same point. This can only happen if

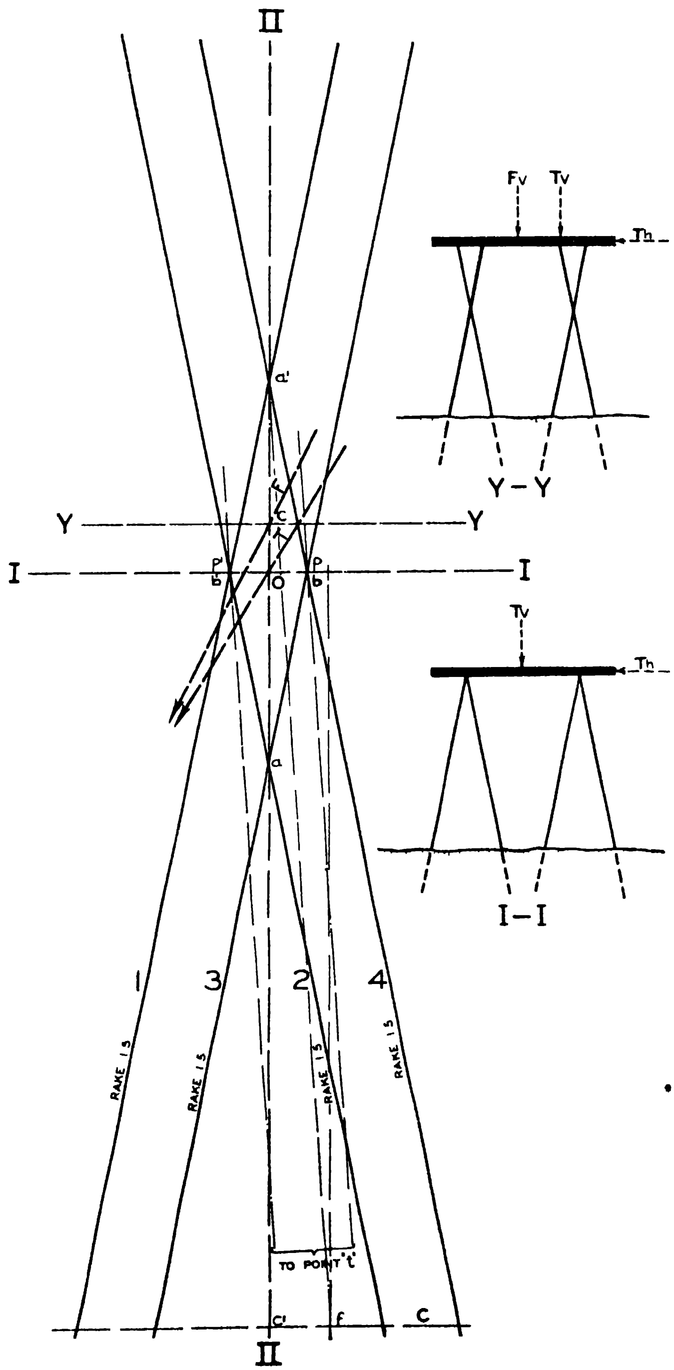
(1) The ratio between the safe carrying capacity of the piles and their resistance against pull is 1 (and not 2 as assumed), in which case t moves to r'. In this case the centre of rotation lies on the vertical axis 1, that is, the corresponding force is horizontal; in other words, the "ideal" vertical load is nil.

(2) Line 1 is parallel to 3, and 2 parallel to 4 (see *Fig. 22*). In this case the points u', r', u and t are points at infinity, p' t is parallel to p t, and all four lines intersect at t, a point at infinity. The corresponding force T must pass through O. The inclination of T indicates the desirable ratio between the vertical and the horizontal load. (This case is, of course, quite simple, and the result might easily be obtained without introducing O-points and the like, but the advantage of the more general method lies in the fact that the effect of any variations of the conditions may be ascertained at a glance.)

Apart from these two cases there is no force which will fully exploit the elastic resources of all the piles simultaneously, but it is practically always possible to find a force which will very nearly do so. Referring to *Fig.* 21 it is seen that if point t or a point close to t is chosen as centre of rotation, the error will be small, since the distance from t to 1 and 3 is practically the same, and so is the distance from t to 2 and 4. The "ideal" force is therefore very nearly the antipolar T of t. T is found by constructing the antipoles v' and w' of a horizontal and a vertical line through t. [$ov \times ov' = (oa)^2$ and $ow \times ow' = (ob)^2$.]

So far it has been found that for any cross section of a jetty with the piles arranged in accordance with the diagram in *Figs.* 21 or 22 (and as shown in *Fig.* 17 this covers quite a number of cases, depending on the position of the deck in relation to the pile axes), the maximum resistance against horizontal forces is obtained when the resultant of all vertical and horizontal forces acts in the line T.

At the moment jetties are being considered which are symmetrical about a vertical axis and which may receive a blow from either side; it is therefore natural to assume that the resultant of the vertical loads T_v acts in the vertical axis I. In that case the horizontal force T_h should strike at the level X–X (*Fig.* 21), which means that the deck should be at this level to ensure the best


II
a'
c
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Y
I
I
p'
b'
O
p
b
a
1
3
2
4
RAKE 1:5
RAKE 1:5
RAKE 1:5
RAKE 1:5
TO POINT 't'
c'
f
c
II
Fv
Tv
Th
Y - Y
Tv
Th
I - I

Fig. 22.

results (assuming, as before, that all horizontal forces acting on the jetty are transferred to the deck through fender piles). This gives section X (*Fig.* 21) as the best solution, which is practically the same result as was obtained without taking the vertical forces into account.

For this section, then, T_h is the maximum horizontal force which can be resisted by the pile trestle without encroaching upon the factor of safety and only so if the trestle simultaneously is subjected to the "ideal" vertical load T_v. If the trestle is struck by a blow corresponding to a horizontal force smaller than T_v, then it would be better if the vertical load were correspondingly decreased, inasmuch as this then would produce the minimum reactions in the piles; but in any case the reactions would not reach the permissible figures, so that it is on the safe side to use the vertical load T_v. On the other hand, if the horizontal force exceeds T_v, the jetty would be safer with a larger vertical load, but the prescribed limit of safety would in any case be overstepped. The designer must, of course, work on certain assumptions, and one of these is to fix a limit for the horizontal blows which the jetty should be expected to resist without encroaching on the factor of safety. The design should then be based on this limit, and the corresponding capacity for resistance obtained in the most economical way.

Considering another section, for instance, Y (*Fig.* 21), with the deck at level Y–Y. The horizontal forces are then applied at this level, and the only possible way of utilizing the piles fully is to let the resultant of all the vertical forces T_v pass through the point e in which T and Y–Y intersect. If this could be done this section would be as efficient as section X; it could resist the same maximum horizontal force and could absorb the same amount of work. The same applies to section Y in *Fig.* 22, if the resultant of all the vertical forces passes through f. The first example is hardly of interest, as the eccentricity of the centre of gravity is unreasonable for practical purposes, but section Y in *Fig.* 22 might be employed under special circumstances. Both these sections, however, and any section where the dead weight of the deck is not symmetrically arranged about the vertical axis I, cannot be usefully employed unless the jetty is only open to horizontal forces on one side. There is then no reason why the piles should be arranged symmetrically; this case would therefore have to be treated separately.

Returning to section Y–Y (*Fig.* 21), and assuming therefore that the vertical loads are arranged symmetrically, this section cannot offer the same resistance to a horizontal blow as section X, but it may be asked, what is the "ideal" vertical load, the load which enables the jetty to withstand the maximum possible horizontal force? This can be found as follows.

The resultant of the vertical and horizontal forces must pass through c, the point of intersection of Y–Y and the vertical axis, since the horizontal force acts in the line Y–Y and the resultant of the vertical forces acts in the axis I. It follows that the instantaneous centre of rotation must lie on the antipolar to c, which is a horizontal line C passing through c', the conjugate point to c. [$oc \times oc^1 = (oa)^2$]. (This follows from the law that a force and its corresponding centre of rotation stand in the relation of polar and antipole to each other, and that if a line passes through the antipole of another line, then this other line passes through the antipole of the first.)

The instantaneous centre of rotation corresponding to the "ideal" force F is a point on C, the distances from which to the two outer pile axes 2 and 3 stand

in the same ratio to each other as the predetermined ratio between the maximum permissible reaction in a compression pile and the corresponding reaction in a tension pile. If this ratio is 2 : 1, then the centre of rotation—which must lie between the outer pile axes to produce compression in one pile and tension in the other—is twice the distance from 2 that it is from 3 if the horizontal force acts from the left-hand side, producing compression in pile No. 2 and tension in No. 3, or it is twice the distance from 3 that it is from 2 if the horizontal force acts from the other side.

This will be proved correct. Assuming that the force acts from the right-hand side, the instantaneous centre is f and the "ideal" force is F. The magnitude of F is determined by the fact that the reactions in the piles have to be kept below, say, 50 tons in compression and 25 tons in tension, and F is the force which just produces 50 tons compression in pile No. 3 and 25 tons pull in pile No. 2. The horizontal component F_h of F is the largest horizontal component of any force passing through c without overstressing the piles; for if we move the instantaneous centre to the right from f, on the line C, then the corresponding force will rotate around point c in a clock-wise direction, finishing by being horizontal when the centre of rotation reaches c^1. This means that the ratio between the vertical and the horizontal component of the force will decrease, but at the same time the horizontal component will also decrease. Were this not so, and assuming it to remain constant while the vertical load decreases, then the pull in pile No. 2, which was 25 tons when the force F acted on the system, would be increased, but this would mean overstepping the limits imposed. If, on the other hand, the instantaneous centre is moved to the left from f, the corresponding force will rotate round c in an anti-clockwise direction, approaching the vertical axis when the centre of rotation recedes to infinity. The ratio between the vertical and the horizontal component of the force will increase, but the horizontal component of the force must decrease. If, for instance, it were to remain constant and the vertical load increases, then the compression in pile No. 3, which was 50 tons when the force F acted, would be increased to a figure above the permissible load.

F_h must therefore be the maximum statical horizontal load which the system can resist, and F_v is the corresponding "ideal" vertical load. The same "ideal" load also enables the structure to withstand the maximum horizontal blow, as during an impact only the horizontal force does any work, and the total work which can be absorbed by a given section is proportionate to the square of the maximum static force which can be resisted.

The "ideal" load may also be found by a mathematical treatment of the equations for the pile reactions (12), CONCRETE AND CONSTRUCTIONAL ENGINEERING, March, 1934, p. 217, and in simple cases, for instance, when all the piles have the same inclination as in *Fig.* 22, this is probably the simplest method.

It has then been found that there is for every pile trestle an "ideal" load which enables the trestle to withstand the maximum possible impact. It is also natural to ask how important is it that this "ideal" load should be provided, and how much does a variation in the vertical load affect the power of resistance against blows. The answer depends upon the type of trestle under consideration, the inclination of the piles, but, above all, on the ratio between the permissible compression and the permissible pull in a pile. If there is to be no pull in the

piles, which would probably be the case if the piles only penetrate very little into the ground, then the trestle could not absorb any horizontal blow unless it were loaded On the other hand, if the permissible figure for the compression and pull is the same the 'ideal" load is nil The answer can, of course, be found in each case by the methods previously given but in order to get an idea of the

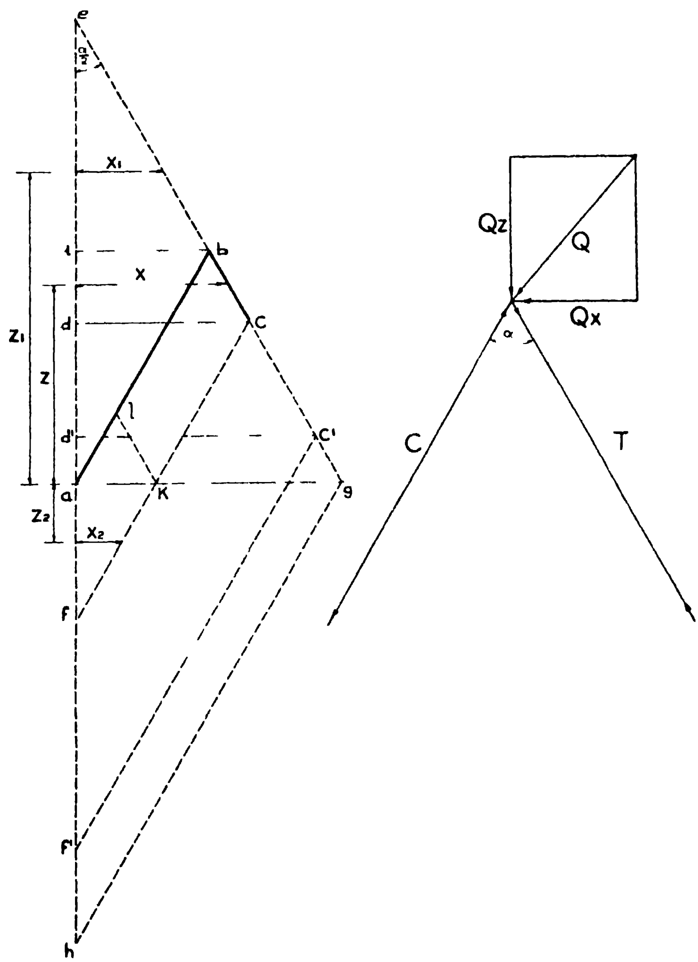

Fig. 23.

significance of a variation in the vertical loading, the simplest possible case, a pile trestle formed by two raking piles with the same inclination and connected at the top (*Fig* 23), will be considered This trestle is subjected to a vertical force Q_z, and Q_x is the largest horizontal force which the trestle can withstand while keeping the reactions in the piles inside the permissible figures of C in compression and T in tension The angle between the pile axes is α

In the force diagram, *Fig* 23, $a\,b = C$ and $b\,c = T$ If these maximum reactions are to be produced in the piles the trestle must be subjected to a vertical

load $Q_z = a\,d$, and a horizontal force $Q_x = d\,c$. $Q_z = a\,d$ is the "ideal" load and $d\,c$ is the maximum horizontal force which the system can withstand. If Q_z is increased or decreased, the corresponding maximum horizontal force Q_x can be found by plotting the distance $Q_z - z$ from a along the vertical line fe and finding Q_x as the horizontal distance X from the point found to the line $e\,c\,f$. If Q_z is increased beyond ai the "tension" pile will always be subject to compression, and when Q_z reaches $a\,e$ the compression in both piles will reach the maximum C and $Q_x = 0$. If $Q_z = 0$ the compression in the first pile is reduced to $a\,l = k\,l = b\,c = T$, and $Q_x = a\,k$. If Q_z is negative Q_x will be further decreased until $Q_x = 0$ for $Q_z = a\,f$, when both piles have the maximum tension T from the start.

The effect of an alteration in the ratio between C and T is also easily seen. If T is increased to $b\,c'$, the "ideal" load is $a\,d'$ and the corresponding horizontal force is increased to $d'c'$. If $T = C$ the "ideal" load is nil and x is measured from $h\,c$ to $e\,g\,h$.

If $\frac{C}{T} = m$, the ratio between the horizontal force which the trestle can withstand when loaded with the "ideal" load and the force it can resist when $Q_z = 0$ can be simply expressed by $\frac{m+1}{2}$. In other words, if a pile can only resist one half the axial force in tension that it resists in compression the efficiency of the trestle will be increased by 50 per cent if it is loaded to the "ideal" figure; if $m = 3$ the efficiency is increased 100 per cent, and so on.

[The application of this theory in the construction of a jetty recently built in the Thames will be given in our next number.—Ed.]

Books Received.

"Analysis of Continuous Frames." By F. B. Russell. San Francisco: Ellison and Russell. [A series of tables for the rapid design of continuous beams and rigid frames, including cases where the moments of inertia are variable throughout the lengths of the members.]

"Mehrstufige Rahmenformeln." Vol. 1. By K. T. Sitte. Braun: Rudolf M. Rohrer. Price R.M. 8.00. [A comparison of methods of designing continuous beams, with notes on the most suitable analysis in each case. The text includes a very large number of fully worked examples covering all arrangements of spans and allowing for variations in moments of inertia.]

"Il Cemento Armato." Vol. 2. By Dr. L. Santarella. Milan: U. Hoepli. Price L.52. [The fourth and revised edition of the author's treatise dealing with reinforced concrete. The present volume deals with the application of theory to the design of domestic and industrial structures.]

"Structural Steelwork." Edinburgh: Redpath, Brown & Co., Ltd. [A supplement to the firm's "Handbook of Structural Steelwork," containing tables of single and compound beams and columns revised in accordance with the most recent recommendations of the British Standards Institution.]

"Le Applicazioni del Cemento nei Fabbricati Rurali." By Dr. L. Gussoni. Milan: U. Hoepli. Price L.12. [The use of plain and reinforced concrete on farms. Working details of suitable structures.]

"Report on Water Retaining Concrete Structures." London: Institution of Structural Engineers. Price 6d. [Recommendations for the design and construction of reservoirs and dams.]

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TWENTY-NINTH YEAR OF PUBLICATION

INDEX

Reservoir at Sevenoaks.

The reservoir illustrated in *Fig* 1 has recently been constructed under a Ministry of Health emergency order to the design of Mr E H Evans, A Inst W E, Engineer to the Sevenoaks Waterworks Company

The reservoir has a capacity of 1,000,000 gallons and is 134 ft square inside The depth of water is 9 ft and there is a freeboard of 12 in between top water level and the soffit of the flat slab roof An 8 in reinforced concrete wall divides the reservoir into two equal portions so

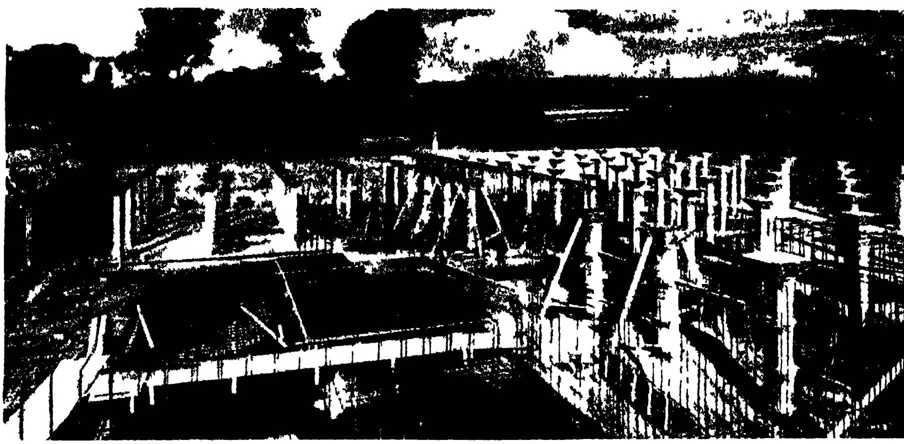


Fig. 1.

as to enable one-half to be cleaned when necessary All walls have a uniform thickness of 8 in with 8 in by 8 in fillets at the bottom The footing of the outer wall projects 10 in outside the bottom edge of the fillet

At one end there is a valve chamber with three 9-in inlet pipes, one from the existing borehole, one from the new bore hole, and the third for a future bore The yield of the existing borehole is 30,000 gallons hourly and its depth is 130 ft In each half of the reservoir there is a sand-retaining pit with baffle walls to the roof, and cross walls 12 in high, giving a passageway 45 ft long The 14-in suction main is carried through to the far end of the reservoir where are also the 12-in washout and overflow In each half of the reservoir there are two manholes and six ventilating shafts There is a 3 ft square manhole over the valve pit

In the 8 in outer walls the reinforcement on each side consists of ⅝ in vertical bars at 6 in centres in the lower part and at 12 in centres in the upper portion where the bending moment decreases The bars on the outer face of the wall are in one length and are bent into the roof slab at the top, there are ½ in horizontal bars on each face at 9 in centres and staggered

The columns are at 12 ft centres in both directions and are 12 in square and reinforced with four ⅞ in vertical bars and ⅜ in hooping Each column has a truncated pyramid cap 2 ft 9 in square and 12 in deep The column reinforcement projects 2 in into the 6 in flat slab roof In typical interior bays the slab is reinforced with six ½ in bars each way alternately bent and straight and six $\frac{7}{16}$-in bars each way alternately bent and straight In addition, there are five ½ in bars each way at 6-in centres in the top of the slab over the column caps

Under the 12-in reinforced concrete floor slab is a 2 in layer of plain concrete The reinforcement in the floor comprises ⅝-in bars at 12 in centres both ways in the top and $\frac{7}{16}$-in bars at 12-in centres both ways in the bottom.

Concrete was mixed in a " Rex 5S "

machine and placed in the walls and columns from runways on staging Timber shuttering was used throughout, and the roof centering was supported on 'Rooshors' The transverse bearers in the illustration (*Fig* 2) are 10 in by 3 in timbers and these carry 7 in by 2 in joists supporting the 1½ in panel forms The latter were made 10 ft long and 6 ft wide

The walls were poured in 4-ft lifts with horizontal grooved joints between each pair and at the level of the top of the

Fig. 2.

fillet Vertical grooved construction joints were also made where required in the wall

The quantities in the reservoir contract were 8,000 cb yd of excavation, 124 cb yd of 2-in plain concrete under the floor, 1 374 cb yd of reinforced concrete, 73 tons of mild steel bars, and 4,600 sq yd of shuttering The reinforcement was supplied and bent by the Indented Bar & Concrete Engineering Co, Ltd, who also prepared the reinforced concrete details The Stone Court Ballast Co, Ltd supplied the coarse aggregate Ordinary Portland cement was used in making the concrete, and the proportions were 1 2 . 4 in all parts of the work

To test the reservoir it will be filled in depths of 3 ft at a time, with intervals of three days between the filling operations The water will be retained in the reservoir for a week before measuring the fall in its surface level

The Engine House.

On the same site an engine house is in course of erection This is also built of reinforced concrete and is finished with concrete paint In plan it measures 45 ft by 42 ft and the height from the engine floor, which is 2 ft 6 in above ground level to the roof is 19 ft A basement below the engine floor accommodates the pipework with the minimum amount of chequer plating The engine room is 42 ft long by 30 ft wide and behind it are a workshop 20 ft by 15 ft, a store 7 ft by 12 ft, an office 10 ft by 15 ft, and a lavatory, all these being above the basement In the latter there is a fuel tank with a capacity of 20 tons The two Ruston 5-cylinder engines will give 220 H P each at 350 r p m, and the Harland pumps will have a discharge of from 40,000 to 80,000 gallons per hour against 400 ft head

Special precautions have been taken to prevent vibration from the engines being transmitted to the building Between

the engine bed and the walls of the engine house the basement floor is a 12-in reinforced concrete raft The isolating material under the engine bed was constructed to the design of Messrs W Christie & Grey, Ltd, using 2 in "Coresil" cork plates A 3-in layer of plain concrete was laid on the 12-in concrete raft extending under the whole building, and covered with building felt On this the "Coresil" plates measuring 6 ft 0 in by 2 ft 11 in were laid and the spaces between were filled with 2 in cork plates Building felt with lap joints was then tacked to the cork and the edges of this were turned up round the shuttering for the bed without cutting The 120 cb yd of mass concrete in the bed was then placed in position

The 6-in reinforced concrete roof slab of the engine house will be carried on two beams having a net cross section of 15 in by 12 in For raising the concrete for the walls and roof an A C E hoist is being used

Messrs Wm Moss & Sons, Ltd, are the contractors for the reservoir and the engine house Mr J C Plumbe, B Sc, is the resident engineer The excavation, sub drainage, and main laying were carried out by direct administration by the Sevenoaks Waterworks Company

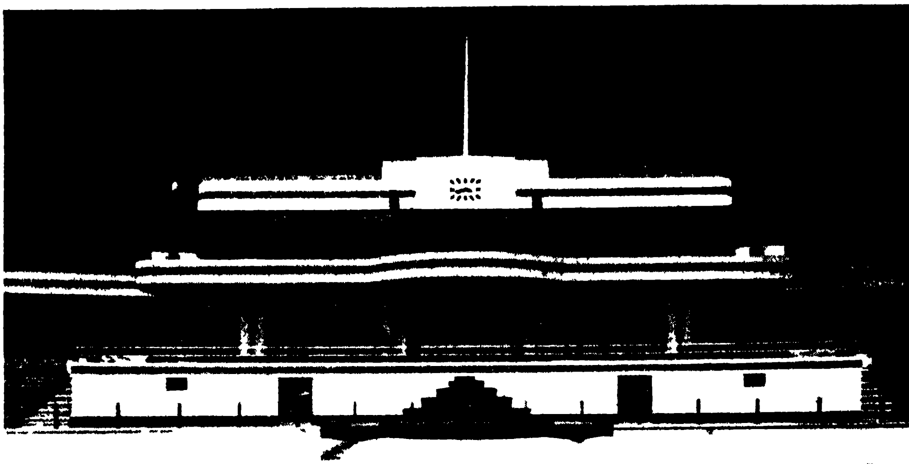
Bathing Pool at New Brighton.

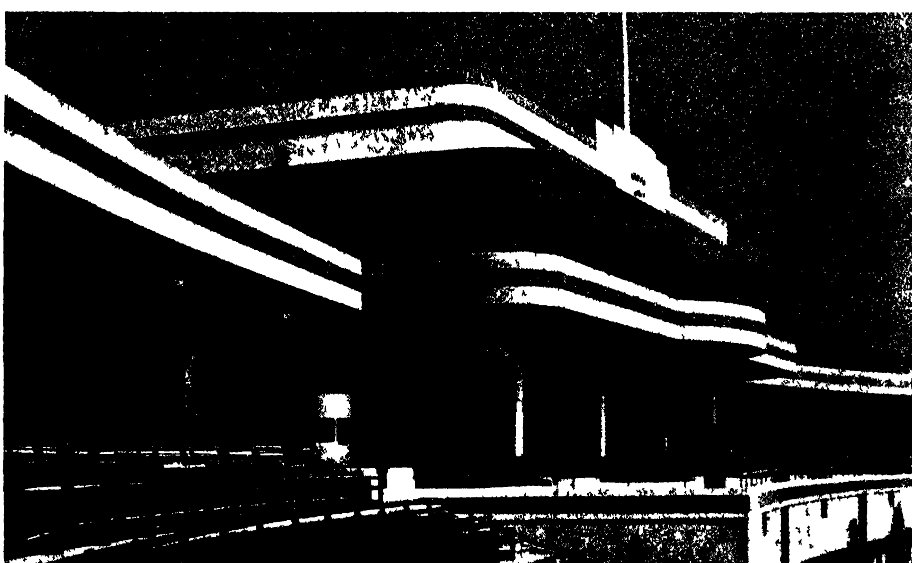
DESIGNED and constructed under the direction of Mr L St G Wilkinson, M C, M Sc, M Inst C E the Borough Engineer of Wallasey, this bathing pool was opened in the summer of 1934 The whole of the terracing and buildings, comprising dressing accommodation, café, administration building, pump room, etc, were constructed in reinforced concrete, the British Reinforced Concrete Engineering Co, Ltd, designing and supplying the reinforcement The surfaces were rendered with white Portland cement, the horizontal emphasis being maintained in green and black faience bands

A feature of the design is the use made of the cantilever in producing overhanging canopies and balconies providing shelter without obstruction, and this is well brought out in the photographs we reproduce The choice of reinforced concrete enabled the building work, commenced during the winter, to be carried out expeditiously Messrs Wm Tomkinson & Son, Ltd, were the contractors.

General View of Pool.

A general view of the pool is given above, while views of the café are given on page 52





Bathing Pool at New Brighton: Two Views of the Café. (*See p* 51.)

Foynes Harbour Extension.

FLAT SLAB DECK ON "NOFRANGO" CYLINDERS.

IN August 1933 a description was published in this journal of the uses of a material known as "Nofrango," which is a fine concrete reinforced with specially manufactured jute, sisal, or other suitable fibre and is claimed to be durable, incombustible, and watertight. In the article referred to brief reference was made to the intention of Messrs. Delap & Waller, MM.Inst C.E., to use "Nofrango" cylinders instead of steel in the construction of the new quay at Foynes, Co. Limerick, so as to take advantage of any possible reduction in cost Particulars of this novel work are now available.

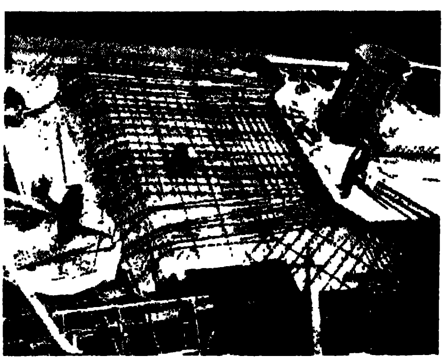


Fig. 1.

Fig. 2.

The pier extension (*Fig.* 3) is approximately 460 ft. long by 50 ft wide, and the decking was designed as a two-way flat slab 14 in. thick with spans of 22-ft. 9-in. and 10-ft. square drop panels A longitudinal beam is provided below decking level at the edge of the slab (*Fig.* 4) to take the impact of ships and transmit the blow to the stiff flat slab (*Fig.* 1) which acts as a horizontal girder. This beam is also used to support the 9-in. diameter round pole larch or Scotch fir fenders, which cost only 2*s.* each and are easily replaced when necessary.

There is no bracing in the sub-structure of the pier, and no half-tide work was required. Reinforced concrete piles were cast in lengths of 54 ft. to 55 ft. and were driven in groups of two or three, as required by the distribution of the loads, to a penetration of about 40 ft. Cylinders of "Nofrango" were then lowered over the groups of piles and filled with mass concrete, on top of which the caps (*Fig.* 2) and drop panels were cast in position to support the decking.

The total number of cylinders is 61; the longest is 37 ft by 6 ft diameter and the smallest 20 ft. by 4 ft 6 in diameter The latter weighs 3 tons The thickness of the cylinders is 1¼ in. and they are reinforced with four layers of jute The cylinders were made on temporary wooden moulds which were collapsed and lifted out, it was found safe to lift the cylinders 5 days after they were finished. They have proved absolutely waterproof, even when there is 30 ft. head of water outside.

A 5-ton loco. crane picks up the cylinder and travels along with it until it comes within range of the derrick (*Fig.* 5) used for pile pitching. The derrick then takes over the cylinder (*Fig.* 6) and lowers it into the bed of the harbour where a small amount of excavation has been grabbed out on the site of each pier The cylinder is allowed to take the bottom and then two piling hammers weighing 4½ tons are placed on top of it as kentledge. As a result of

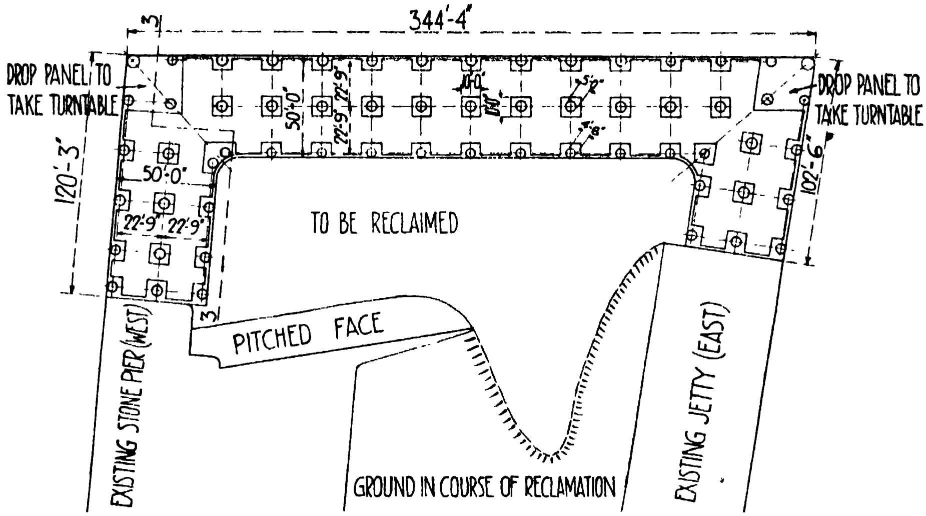
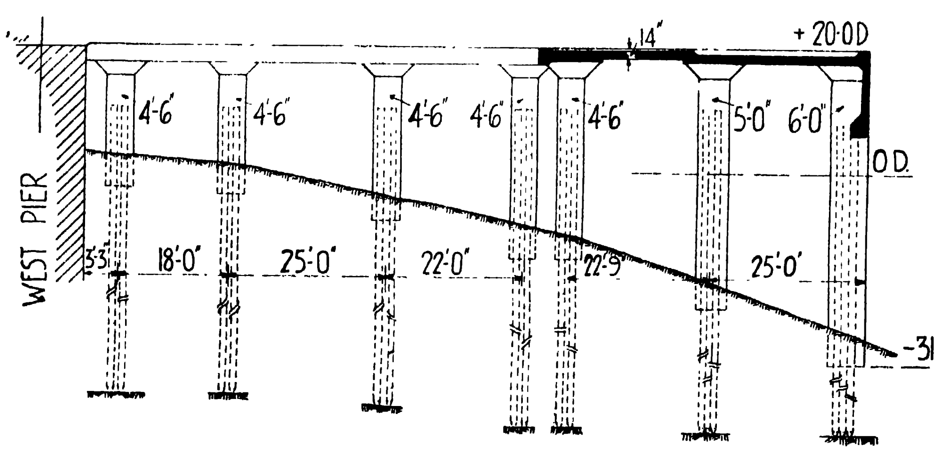


Fig. 3.



SECTION-ELEVATION ON LINE 3-3

Fig. 4.

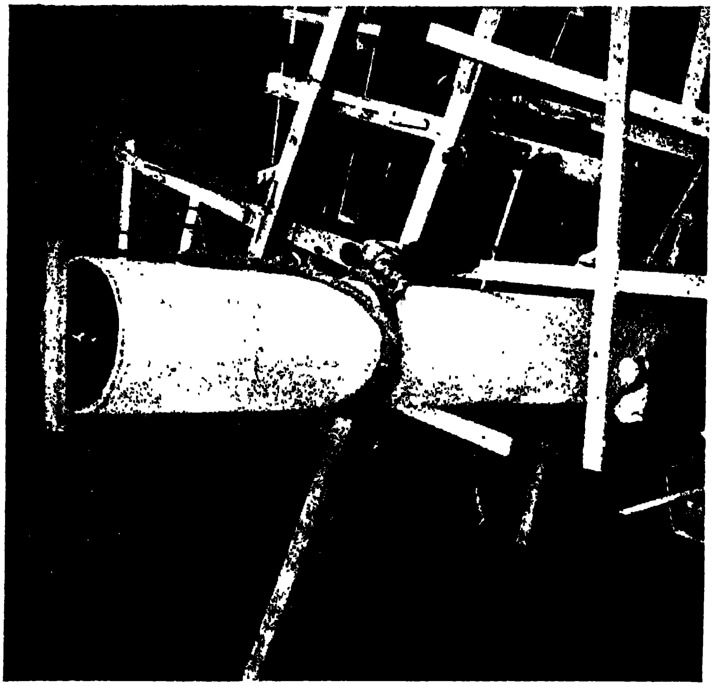


Fig. 6.

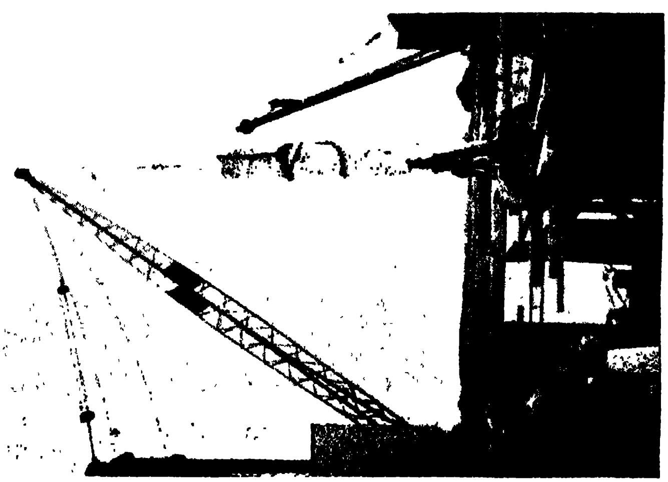


Fig. 5.

this kentledge the cylinders sink from 1 ft 6 in to 2 ft into the mud after which they are pumped out dry and filled with solid concrete.

In order that the cylinders may be filled rapidly they are strapped every 3 ft with a hoop iron band. This was not done in the case of the first twelve cylinders but was introduced as an extra precaution to cover any error in estimating the pressures that come on a high cylinder when it is filled with wet concrete by means of a dump bucket.

The contractors are Messrs. McCaffery and O'Carroll, of Limerick, for whom Mr. T. A. Simington, B.A.I., is agent. Mr. R. F. Waller, M.Inst.C.E., is the resident engineer.

Extension to Parkeston Quay, Harwich.

This work comprises the extension and widening of the existing London & North Eastern Railway quay at Parkeston and the construction of two approach viaducts. The new quay is 1,120 ft long by 140 ft wide and provides three additional steamer berths. A length of 450 ft at the rear of the existing quay was widened by 14 ft 6 in to accommodate a roofed platform for the customs examination of train ferry wagons.

The new quay is built as a series of main trestles, spaced at 25-ft centres and carried on 16-in by 16-in and 14-in by 14-in precast piles with intermediate trestles at 12-ft 6-in centres. The trestles are connected at their tops by the decking and deck beams, and at a lower level by bracings in a horizontal plane. The front four rows of piles are 16 in by 16 in and the remainder 14 in by 14 in, all reinforced with 1⅛-in diameter mild steel bars. Each main trestle is fronted with a pair of 16-in by 16-in piles spaced 10 in apart and encased with a 6 ft diameter precast concrete cylinder filled with concrete, and the whole made monolithic with the junctions of piles and bracings. All the piles were cast in lengths of 38 ft 6 in or 40 ft and were driven to a set of 1 in for the last 15 blows of a 3 ton monkey with a 2 ft 6 in drop. This part of the work is generally illustrated in *Fig.* 1, which is a view looking west taken in January 1933.

Fig. 1.

The deck slab is 6 in thick and designed for a loading on the railway tracks equivalent to 18 B.S. units of loading, corresponding to 100 lb per square foot in addition to dead load.

The North Viaduct (*Fig.* 2) is 27 ft wide by 537 ft long and is built on a curve of 10 chains radius. It carries two tracks serving the goods roads on the front of the main quay. The design load of the deck is 18 B.S. units. The deck slab is 6 in thick with main transverse beams 12 in by 24 in and longitudinal

beams 9 in. by 13 in. spaced at 3-ft. 10-in. centres.

The South Viaduct (*Fig.* 3) is 16 ft. wide by 498 ft. long; it has a radius of 11½ chains, and carries a single-line passenger track. The decking, designed for spaced at 12-ft 6-in. centres measured on the centre line of the curve and disposed radially. The precast piles vary in length from 37 ft. to 43 ft. The cost of the reinforced concrete work was approximately £120,000. The whole of the

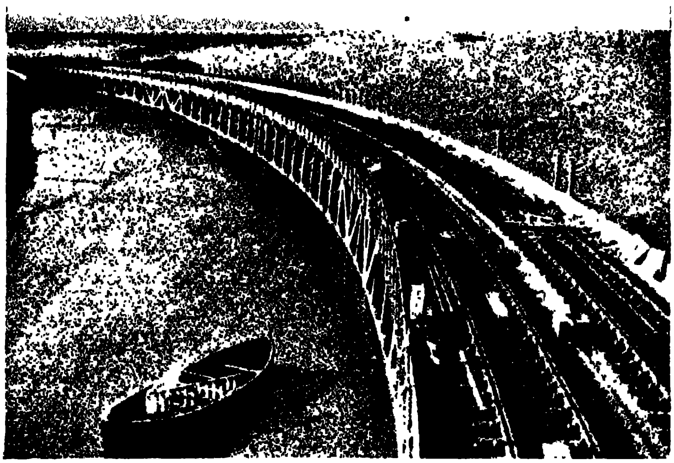


Fig. 2.

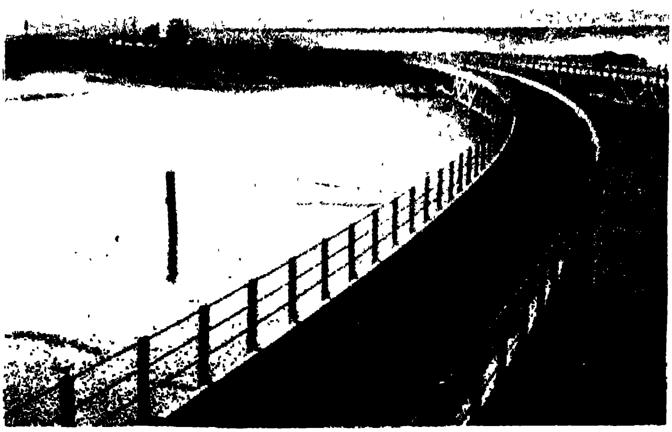


Fig. 3.

the same loading as that carrying the passenger track on the main quay, consists of a 6-in. slab with 12-in. by 26-in. transverse beams and 9-in. by 13-in. longitudinal beams at 3-ft. 10-in. centres.

The trestles in both viaducts are works were carried out to the design and under the supervision of Mr. Chas. J. Brown, Chief Engineer Southern Area, London & North Eastern Railway Co., by the Yorkshire Hennebique Contracting Co., Ltd.

Berthing Arm at Clacton Pier.

THIS extension to Clacton pier (*Fig.* 1) provides berthing accommodation for pleasure steamers. It has an overall length of 315 ft. and a width of 30 ft., and accommodates three ships. Landing platforms of Jarrah timber have been constructed at 9 ft. 3 in below top deck level for the use of passengers at low tide. At high tide the deck is 6 ft above the water; the total height above the sea bed is 30 ft.

The main structure is constructed entirely in reinforced concrete and is supported on 96 piles, 15 in square by 42 ft. long, driven to a rake of 1 in 5 and arranged in trestles of four piles (*Fig.* 2) tied together at the top by a cross beam; the trestles are spaced generally at 15-ft. centres. This trestle arrangement provides great resistance to shocks from berthing vessels, the lateral forces being resolved into direct compression and tension forces in the piles. If we consider the forces acting on one pair of raking piles, let W be the available dead load, P the side thrust, and C and T the maximum compression and tension forces allowable in each pile,

then $$W\sqrt{\frac{26}{10}} - P\sqrt{\frac{26}{2}} = T$$

and $$W\sqrt{\frac{26}{10}} + P\sqrt{\frac{26}{2}} = C.$$

From these equations it will be seen that the amount of side thrust P capable of being taken up by the piles for a given dead load W is limited by the capacity of the piles to take tension. Therefore, to increase the dead weight of the berth and consequently its lateral resistance a novel arrangement was resorted to.

The end of the berth was constructed with closely-spaced raking pile trestles consisting of 22 piles surmounted by a reinforced concrete box (*Fig.* 3), 19 ft. long by 30 ft. wide by 8 ft. deep, filled with 280 tons of stone.

Fig. 1.

An impact on any part of the berthing arm will be transmitted through the deck to the trestles and ultimately to the strengthened end. The decking acts as a horizontal girder with the edge beams as compression and tension flanges, and for this purpose the edge beams are heavily reinforced.

The main fender piles consist of two 14-in. square baulks of greenheart and turpentine bolted together and faced with brushbox rubbing strips. The tops of the fender piles are arranged in vertical grooves set in the concrete edge beams. Built into the concrete behind the grooves are spring buffers capable of taking a pressure of 30 tons for a compression of 2½ in. The more sheltered parts of the berth are provided with single piles of greenheart or turpentine timber, 16 in. square, and

the timber walings are of creosoted Oregon pine

The work was executed under difficult cantilevered out first from the existing pier and afterwards pushed on to the staging piles already driven As the

Fig. 2.

weather conditions and was completed in just under six months Timber staging piles were first driven from a frame staging was completed the pile frame was erected and moved out to drive the raking piles through the sea bottom into

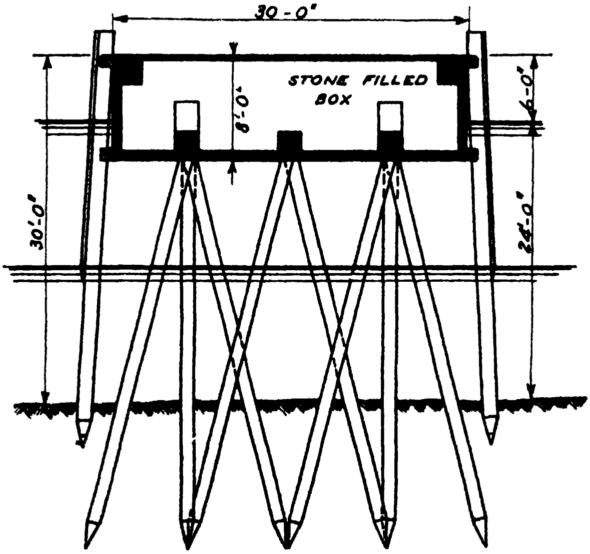


Fig. 3.

hard Platimore sand The piles weigh nearly $4\frac{1}{2}$ tons each The centering was then erected for the deck and beams

The deck was fitted with seven Bean type bollards At the outer end a signal mast 50 ft high was erected the lower 20 ft being constructed of reinforced concrete and the rest in timber, made in such a way that it can be quickly lowered to the deck

The work was designed and carried out by Messrs Christiani & Nielsen, Ltd

Coal Bunkers at Accrington.

The two bunkers shown in *Fig* 1 recently erected at Accrington are equipped with weighbridge platform larry deck and working house and have capacities of 600 and 900 tons of coal respectively Their overall height is 110 ft 1 in The plan dimensions are 36 ft by 31 ft overall for the 600 ton bunker

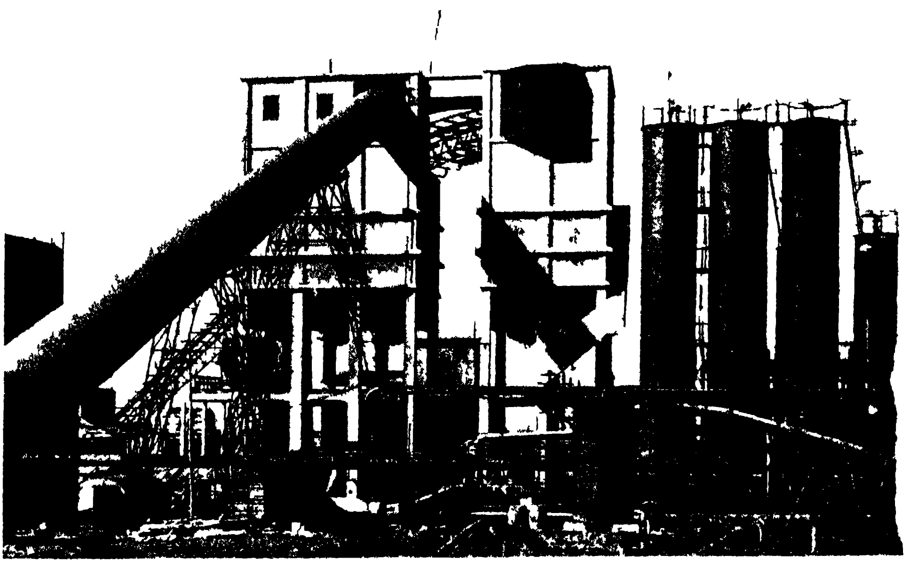


Fig. 1

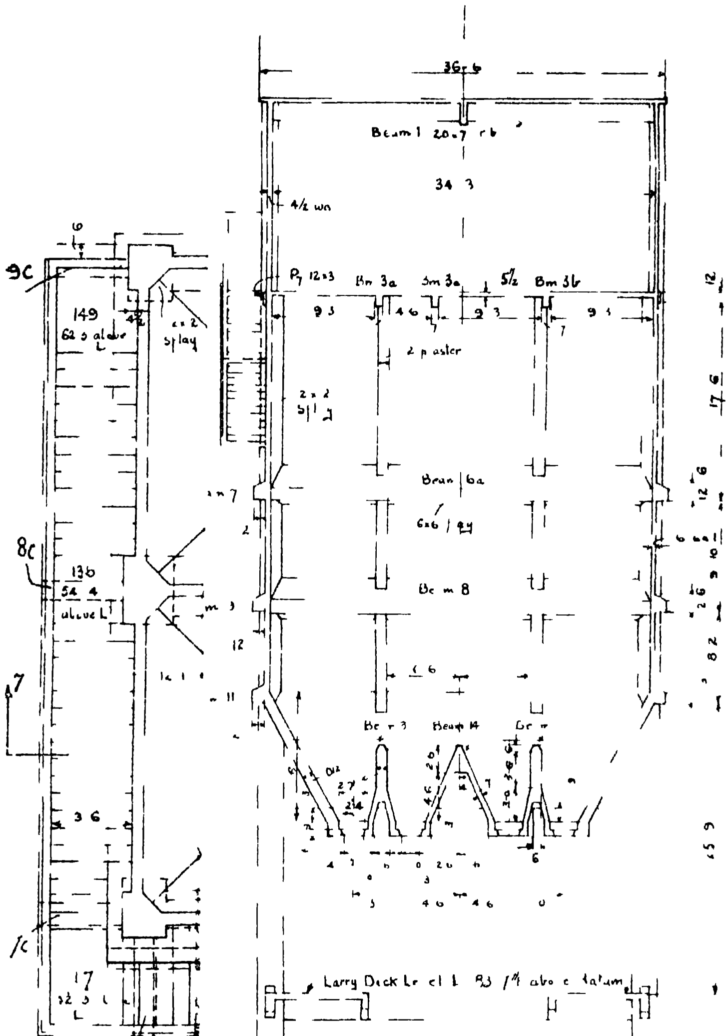
and 36 ft by 45 ft 6 in for the 900 ton bunker A plan and cross sections of the 600 ton bunker are illustrated in *Fig* 2 and details of the hoppers are shown in *Fig* 3 The designs were prepared by Mr A S Grunspan, B Sc M Inst C E

As the larry deck had to be kept clear of intermediate columns, beams marked Nos 15 and 16 (*Fig* 2) had to span about 34 ft The cross hoppers were made use of and these were carried beyond the main hopper slopes, as in beam No 13, to tie into beams marked Nos 15 and 16 Beams Nos 15 and 16 so far as horizontal thrust was concerned, were thus reduced to a span of about 11 ft making a narrow beam possible (*Fig* 4)

The combination of pilasters and beams (*Fig* 5) gives the bunker a pleasing appearance, a factor not to be overlooked in structures of a purely industrial nature

The bunkers are actually filled with 750 and 1050 tons of coal The total quantities of concrete and steel for the 900 ton bunker were 1300 tons and 84 tons respectively

Figs 2, 3, 4 and 5 are given on the following folders The work was carried out by the Monolithic & General Construction Co , Ltd


36 6
Beam 1 20x7 rb
34 3
4½ wn
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149
62 5 above L
splay
Bm 3a
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2 plaster
Beam 16a
Bm 8
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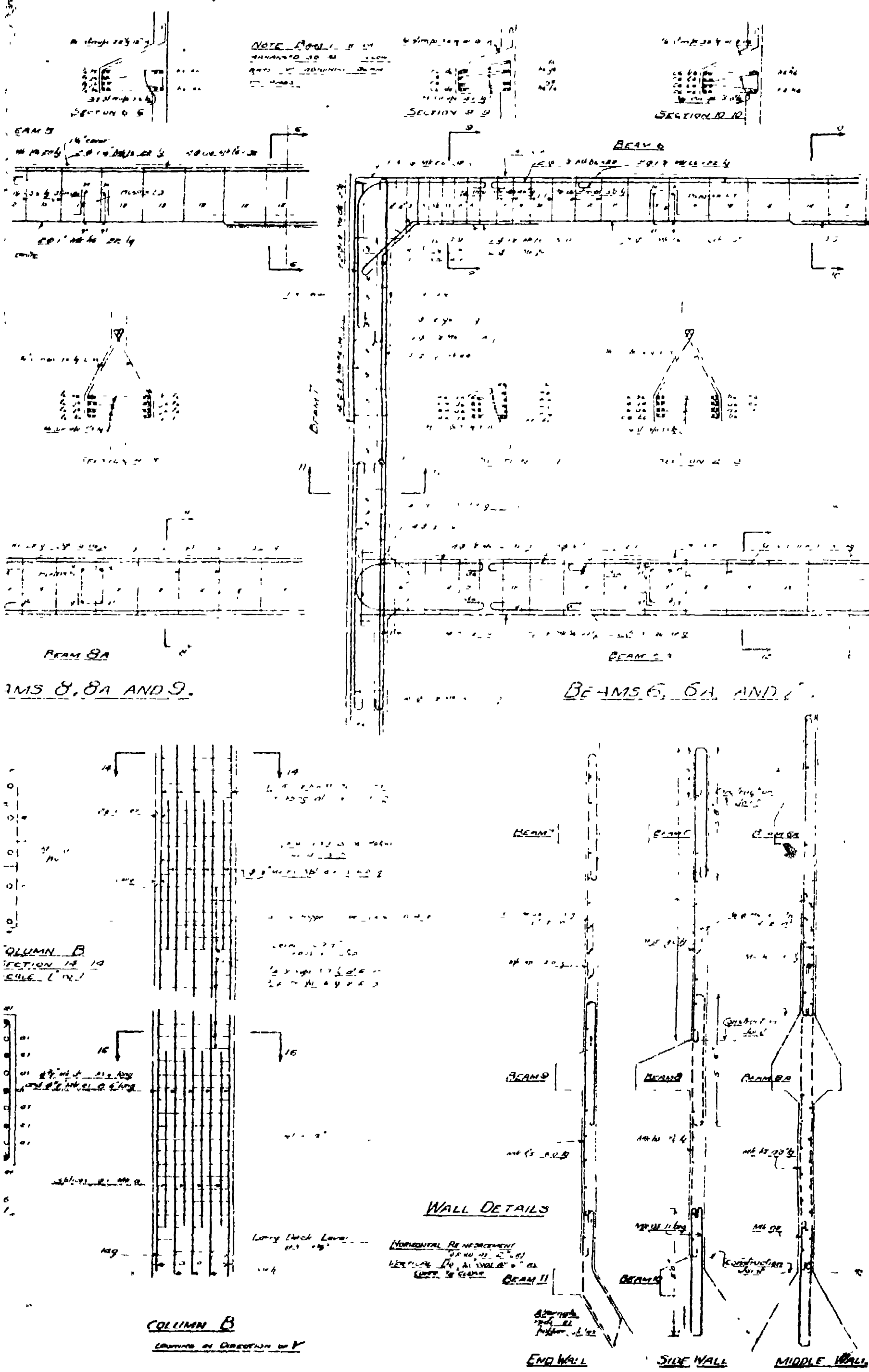
6C
Larry Deck Level 1 93 1¼ above datum

SECTION 7-7



LOPE. (See page 60.)

Factory at Homerton.

THE reinforced concrete building illustrated in *Fig* 1 is a new factory for Messrs Lewis Berger & Sons Ltd which has been erected to the design of Messrs Henry Tanner FFRIBA The contractors were Messrs Trollope & Colls Ltd and the detailed drawings of the reinforced concrete were prepared by Messrs L G Mouchel & Partners Ltd

The building has a frontage of 105 ft to Morning Lane and is 126 ft deep with a total floor area of 50 000 sq ft In plan it is channel shape, each of the wings on one side being 42 ft 6 in long by 52 ft wide From ground floor level to the top of the parapet around the flat roof is 43 ft 2½ in There are three floors above ground level with story heights of 11 ft 1½ in, 10 ft 1½ in 9 ft and 9 ft 5½ in In addition there is a basement 9 ft high

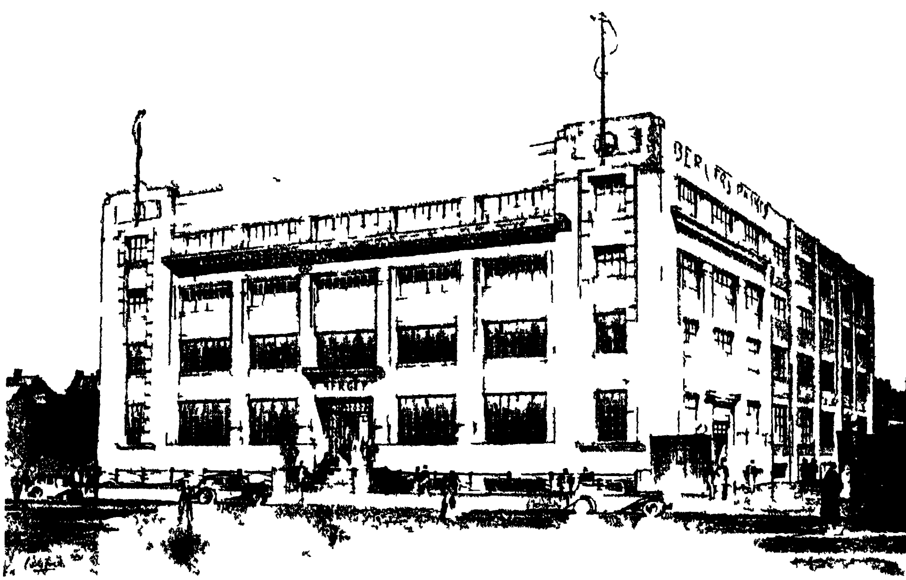


Fig 1

Design.

In the direction parallel to the front elevation the columns are spaced at 14-ft 9-in centres In the rectangular direction the spacings are three at 14 ft 2 in, two at 17 ft 6 in and three at 14 ft 2 in The main beams have spans of 14 ft 9 in and are intersected at their middle points by secondary beams with spans of 14 ft 2 in and 17 ft 6 in Exclusive of the slab thickness the dimensions of the main beams are 14 in deep by 8 in wide while the secondary beams are 10 in deep and 6 in wide In the former the tensile reinforcement at midspan consists of four bars in two layers in the latter there are two bars in the same vertical plane The reinforced concrete floor slabs are 4 in thick while the 3½ in roof slab is carried by 12 in by 8 in main beams and 10 in by 5 in secondaries The spandrel beams connecting the exterior columns are 12 in wide and 4 ft 6 in deep they extend 3 ft above the level of the concrete floor slabs At roof level the spandrel beam forms a parapet 15 in wide and 3 ft 9 in high, above the main entrance the total depth of the beam is increased to 7 ft 9 in to give a 6 ft parapet Exterior columns are rectangular in cross section and interior columns are octagonal, the widths of the latter across the flats

being 18 in., 15 in., 14 in., 12 in., and 12 in. respectively in the various stories.

Over the main entrance there is a reinforced concrete canopy 14 ft. 2 in. long with an overhang of 2 ft. 4 in. The canopy is 12 in. thick and is monolithic with the spandrel beam of the first floor. A detail is shown in *Fig. 2.*

Finishes.

The exterior of the reinforced concrete front elevations and two returns was finished with Messrs. Berger's Matroil, a washable oil bound water paint, in a buff shade. The remaining elevations are in reinforced concrete with London stock brick curtain walls. The interior walls and ceilings have been painted with Matroil "sunshine yellow" with dados in Berger Pompeian enamel paint of a light tan colour. Magnesite composition, in a shade to match the dado, is used as a floor finish.

There are two reinforced concrete staircases leading to the roof and provided with concrete hoods, that at the main entrance being finished in terrazzo and the other in granolithic. In the entrance hall and on the main staircase the walls have been finished with tiles to a height of 5 ft. 6 in. above the terrazzo skirting. The roof carries the lift machinery and supports a steel tank on reinforced concrete bearers.

Construction and Concrete Proportions.

In constructing the foundations for the columns mass concrete blocks 7 ft. 6 in. square and 3 ft. deep were cast in situ and covered with a layer of asphalt. On the latter the 4-ft. square by 2-ft. deep reinforced concrete footings were constructed and surrounded by asphalt which was afterwards connected to the asphalt in the basement floor, thus forming a completely enclosed basement tank.

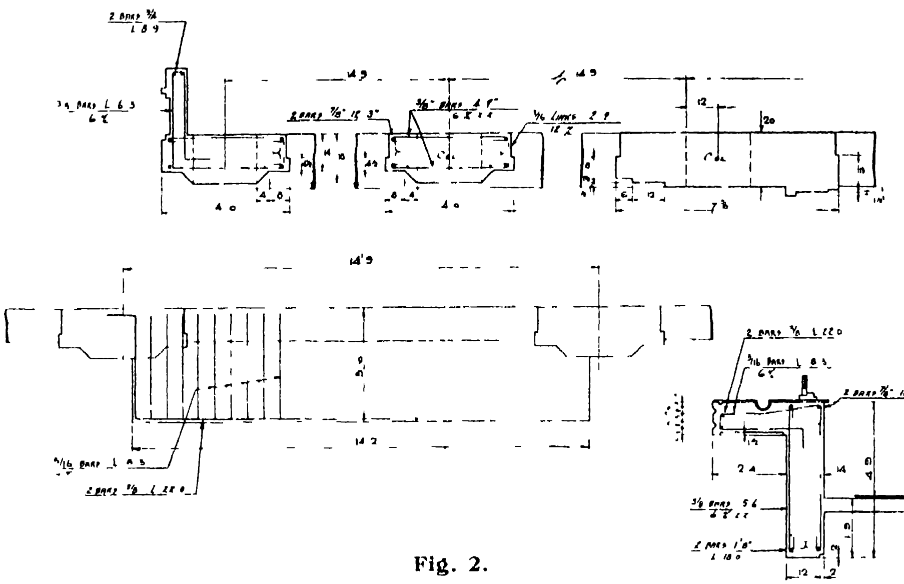


Fig. 2.

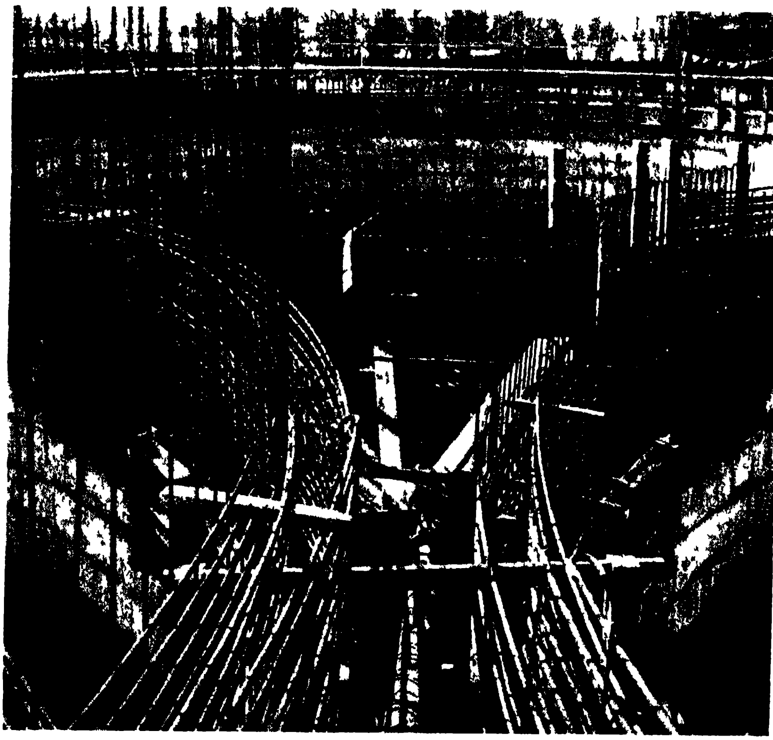
The basement has a 6-in exterior reinforced concrete wall, a $1\frac{1}{2}$-in. asphalt layer, and a 6-in internal reinforced concrete wall. The 6-in. reinforced concrete floor slab was laid on 3 in. of plain concrete covered with asphalt.

Coarse aggregate not exceeding 2 in. in size was used in the concrete for the foundations, and $\frac{3}{4}$-in. flint chippings were used in the reinforced work, except on the front elevation where the maximum size of the aggregate was $\frac{3}{8}$ in. In the reinforced work and the concrete for the tank the proportions used were 6 cwt. of Portland cement to $13\frac{1}{2}$ cb. ft. of sand and 27 cb. ft. of coarse aggregate. For the mass concrete below the footings the proportion of cement in the concrete was reduced to 4 cwt. to 27 cb. ft. of ballast passing a 2-in. ring. The aggregates were supplied by the St. Mary's Wharf Cartage Co., Ltd.

Sewerage Works at Dartford.

THE extensions which are now approaching completion at the Long Reach works of the West Kent Main Sewerage Board afford typical examples of the application of reinforced concrete to modern sewage works practice. The scheme comprises a set of three sludge digester tanks with a power station, boiler house, booster house and foundations to a gasholder and purification plant. The tanks, 85 ft. diameter and 29 ft. deep, are arranged in clover leaf formation with the inter space used as a sludge well, and are covered by a reinforced concrete gastight double roof resting on a structural steel framework. The seal of the roof, which rests on the walls, is formed by a copper strip joint.

The power station, in which provision is made for future extension, is a complete reinforced concrete structure with a clear roof span of 43 ft., the frames being at 12 in. centres. The flat roof forms a cooling pond for the water used by the two 250 h.p. gas engines which form the main equipment of the station.



Digester Tank during Construction.

In view of the nature of the site, the whole of the works were carried on over 700 West Rotinoff piles 15½ in. diameter. The entire construction was carried out to the designs of the British Reinforced Concrete Engineering Co., Ltd., under the instructions of the Chief Engineer of the Board, Mr. A. E. Scott Murray. The contractors are Messrs. A. Jackaman & Son, Ltd.

Lining and Waterproofing a Reservoir with Gunite and Asphalt.

THE Prestwich No. 2 Reservoir of the Manchester Corporation was constructed about 1867 and has a capacity of approximately 30 000 000 gallons. The bottom had been lined with brick and the slopes had been pitched. Certain portions of the pitching had from time to time been concreted or treated with cement slurry. *Fig.* 1 shows the condition of the slopes prior to the new waterproof lining.

The floor was lined with ¾ in. of asphalt, in accordance with the specification of the Natural Asphalt Mine Owners and Manufacturers Council, in which Trinidad Lake asphalt was an integral part. On the upper 17 ft. of the slopes, unreinforced gunite 1 in. thick was applied; on the remainder of the slopes the gunite was applied to a thickness of not less than 2 in. and was reinforced with B.R.C. electrically welded fabric.

At all corners and points where the slope changes materially the gunite is 4 in. thick for a distance of 4 ft. to 6 ft. wide and contains two layers of reinforcement; the shooting of the double thickness at a corner is shown in *Fig.* 2.

The total area of the lining placed under this contract is 247,500 super feet and the contract price was approximately £8 250. The work was commenced about December 1, 1933, carried on continuously during the winter, and completed early in May 1934. The work was executed by the Cement Gun Co., Ltd.; the Limmer & Trinidad Lake Asphalt Co., Ltd., were sub-contractors for the asphalt work.

Fig. 1.

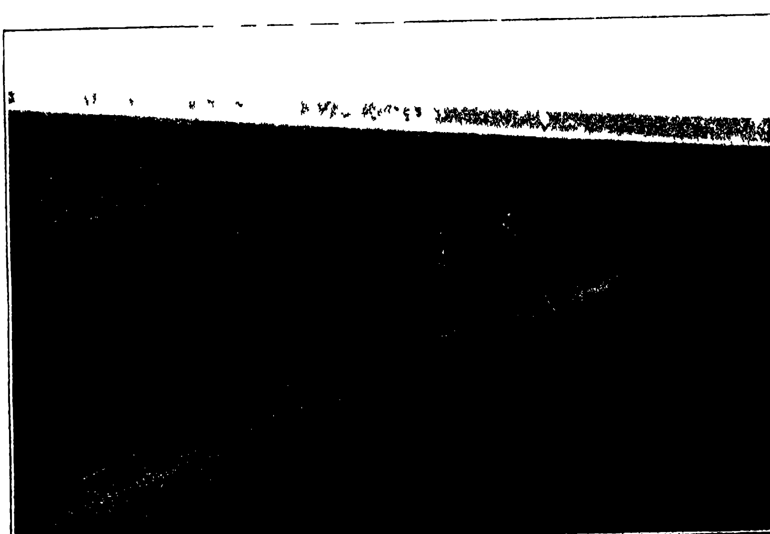


Fig. 2.

Grain Silos at Avonmouth.

These silos were completed last year for Messrs R & W Paul Ltd The total capacity is 5,500 tons of grain which is stored in 21 bins each 11 ft square and generally 100 ft high Four of the bins are divided into three compartments to serve as delivery bins The bins are covered with a reinforced concrete floor over which is a flat concrete roof The overall dimensions of the building are 98 ft 6 in by 36 ft by 132 ft to the main roof, the tower extending to a further height of 12 ft The foundations consist of 236 pre-cast reinforced concrete piles 50 ft long, two piling frames and a 5-ton travelling steam derrick were used

One half-yard mixer, discharging directly into the bucket of an Insley mast hoist plant from which concrete was taken by barrows was used for concreting the superstructure

Steel scaffolding was erected all round the building and inside the bins with sloping gangways in the outside scaffolding as shown in the illustration An elevator tower was constructed at one end of the building Timber forms were used, a single lift 3 ft high being fitted over the whole area After concreting each 3 ft lift the forms were struck and refixed at the higher level by means of special clamps This method did not require any ties through the walls and was economical in labour and materials

The surfaces of the concrete were rubbed down immediately the formwork was

Grain Silos at Avonmouth

struck, leaving a finish which did not require subsequent treatment

An extension of the same contract consisted of piling for foundations for a new mill involving 236 precast reinforced concrete piles 14 in by 14 in by 50 ft long, some of which were extended to 66 ft and re driven

The architects for the work are Messrs John Clarke & Son, F/L R I B A , the reinforced concrete designers are Messrs L G Mouchel & Partners, Ltd , and the contractors Messrs Stewart & Partners, Ltd The steel scaffolding was supplied and erected by Scaffolding (Great Britain), Ltd.

Crude-Tar Storage Tank at Wandsworth.

The large crude tar storage tank described was constructed during the winter of 1933 4 at a new tar distillery at Wandsworth for Messrs Crow Catchpole & Co Ltd Built in the bed of a disused canal the tank is 150 ft long by 75 ft wide it is divided into one large and two small compartments, and has a total storage capacity of approximately 800,000 gallons Photographs taken during the construction of the tank are reproduced below and on the facing page

The floor of the tank consists of a reinforced concrete slab, founded on ballast and laid to the same levels as the underside of the footings of the existing brick walling along each bank of the canal In one direction the slab is continued for 120 ft along the canal bed to form a foundation for two large cast iron tanks A system of agricultural drains radiating from a concrete sump in the bed of the canal allowed the subsurface water to be kept down to foundation level during progress of the work, and later the walls of the sump were carried up to the level of the banks to allow the subsurface water level to be controlled by pumping should this be necessary in future

As the existing walls along each bank were in good condition, concrete slabbing 6 in thick was cast against the inside faces to form the sides of the tank, so that only the end and division walls, which span between floor and roof, required to be designed for their full load The cover slab is supported by an arrangement of beams and columns of conventional design but of somewhat heavy construction The cover is required for storage purposes, and it was estimated

Tar Storage Tank at Wandsworth.

that the loads to be carried would be the equivalent of a uniformly distributed superimposed load of 6 cwt. per square foot. The work was designed by the Expanded Metal Company, Ltd., who supplied the reinforcement and supervised the construction. The contractors were The Walker-Weston Co., Ltd.

Repair of a Coke Hopper with Gunite.

A TYPICAL example of the cement gun method of repairing and strengthening reinforced concrete is seen in the repair of a coke hopper at the works of the Wakefield Gaslight Company. The hopper, which was constructed during the war period, recently gave grounds for concern, as patches of the interior concrete surface, cracked by the heat of occasional coke fires, flaked off and left the reinforcement exposed. The chemicals present in the coke caused rapid corrosion to take place, and a number of the bars were corroded right through. The sloping concrete walls were then worn down by the abrasion of the sliding coke, and in extreme cases only a thin outer skin was left. Deterioration had also occurred in the main supporting beams through the concrete top breaking away, so that the compression reinforcement was without protection from corrosion and abrasion.

The engineer and manager, Mr. C. W. Ward, A.M.Inst.C.E., had repairs put in hand by means of the cement gun. All disintegrated material was first carefully chiselled out with light pneumatic hammers. New reinforcement was inserted to take the place of the corroded bars, and the depressions were then filled with gunite to the line of the original surface, as much as 4 in. of material being required in places. The main beams were next examined, and reconditioned one by one on similar lines. Finally the whole interior of the structure was cleaned down and roughened for bond, and the hopper was resurfaced throughout with a 2 in. layer of 3 : 1 gunite, which embodied a special square mesh reinforcement to prevent cracking under the range of temperature which obtains.

These repairs have restored the structure to a new condition at a fraction of the cost of demolition and reconstruction. Not only have defects been effectively remedied, but the new gunite wearing surface, applied under considerable pressure, is a far more durable material than that of which the hopper was originally built. The work was carried out by Messrs. Whitley Moran & Co., Ltd.

Tar Storage Tank at Wandsworth.

(*See page* 66.)

Foundations at Hydrogenation Plant.

EXTENSIVE foundation work has been carried out recently by Messrs. Brims & Co., Ltd., of Newcastle-on-Tyne, for the new plant just completed at Billingham-on-Tees by the Imperial Chemical Industries, Ltd., for their "oil from coal" process. Vibro cast-in-situ piles up to forged steel drums used in the hydrogenation process. *Fig.* 1 shows this crane and its track, 460 yd. long, consisting of two large reinforced concrete capping-beams resting on Vibro piles and each carrying a pair of 112-lb. F.B. rails, reinforced concrete tie beams at 60-ft.

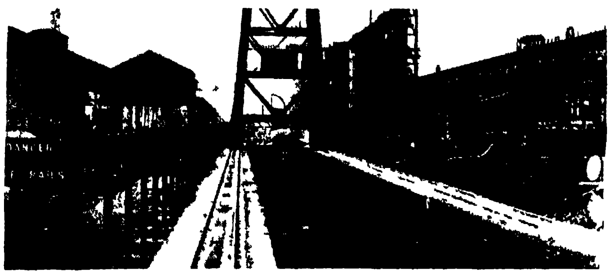


Fig. 1. Crane Track at Billingham.

54 ft. in length are being driven by five 70-ft. frames. Over 2,300 of these piles, under the British Steel Piling Co.'s patented system, were driven on this site, in addition to numerous precast concrete piles required at special places.

Practically all the buildings and structures are carried on piles, including the 170 ton Titan crane which handles the centres connect the two capping beams which are flush with the ground and straight and level throughout.

The contractors handed over a completed portion of track to the crane erectors, Sir Wm. Arrol & Co., Ltd., in $3\frac{1}{2}$ months from starting work, and finished the whole 460 yd. of track in 7 months.

Flats and Garages, Manor Fields, Putney.

SOME interesting reinforced concrete work has been embodied in Manor Fields Estate, Putney, for which Messrs John Laing & Son, Ltd, are the contractors and Messrs Coleridge, Jennings & Soimenow the architects. The estate comprises 12 acres of gardens, fourteen blocks containing 232 flats, over 100 garages, and more than a mile of concrete roads.

In order to economise the space occupied by garages some of these have been built in two decks with the lower ones below ground, the concrete roof of one deck forming the floor of the garages above. Another block of 60 garages is built partially below ground level and the complete building, including underground roadways and turning spaces, is roofed with a reinforced concrete slab 5 in thick, 151 ft long and 124 ft wide. This slab is supported at 10-ft centres by walls or beams, and is reinforced with Twisteel. The 24-ft roadways which run between the lines of garages are spanned by forty concrete beams. With the exception of a few beams subject to heavier loading, these are 9 in by 23 in in section, reinforced with eight 1-in rods in two rows in the bottom and two $\frac{3}{8}$ in rods in the top, with $\frac{1}{4}$ in diameter stirrups. This work was designed by Messrs Twisteel Reinforcements Ltd. The top of the concrete slab is covered with asphalt and provides space for three hard tennis courts with a run back of 22 ft at each end.

One of the Blocks of Flats.

Garages beneath Tennis Courts.

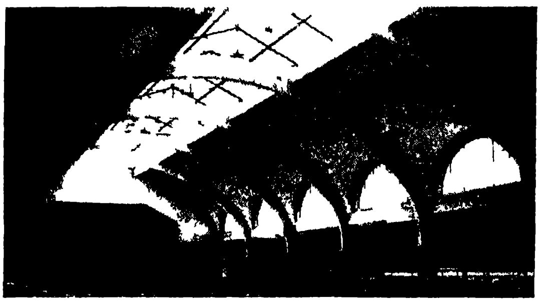
In addition to the concrete roads and garages, reinforced concrete is largely used in the blocks of flats, the floors and beams being constructed in reinforced concrete, as are the roofs, mansard slopes, and dormer windows. The bay windows in the lounge of each flat are supported by a cranked cantilever beam also in reinforced concrete. The total value of the estate is about £500,000.

Canteen Building at Luton.

The whole of the superstructure of this building at Luton is in reinforced concrete, the span of the roof being 72 ft. in the clear and the length of the main hall 168 ft. On each side of the main hall are side wings, also of reinforced concrete, each having a span of 56 ft. The ribs of the roof are designed as free jointed at ground level with a tie beam under the floor; the columns are carried down to the chalk. As the wing walls are of a light nature it was not deemed advisable to use these to take any of the thrust from the main building, and the beams in the main hall were therefore designed as self-contained members. The roof between these beams is only 4½ in. thick, reinforced with Johnson's fabric, the general reinforcements consisting of round steel bars and links. Three-ply wood was used on the shuttering so that plaster would not be necessary, and the resulting surface was found to give a suitable finish when distempered.

The foundation work, basements, etc., were commenced on December 15, 1933, and brought to ground level in January, 1934. The main structure from ground level and the side wings were completed in twelve weeks, notwithstanding rather severe weather conditions. Rapid-hardening Portland cement was used throughout. When the temperature was at freezing point and below the mixing water was heated and the work was covered immediately it was placed.

Messrs. L. Howard & Partners are the architects; the reinforced concrete design is by Johnson's Reinforced Concrete Engineering Co., Ltd., and the work was carried out by Concrete Structures, Ltd.



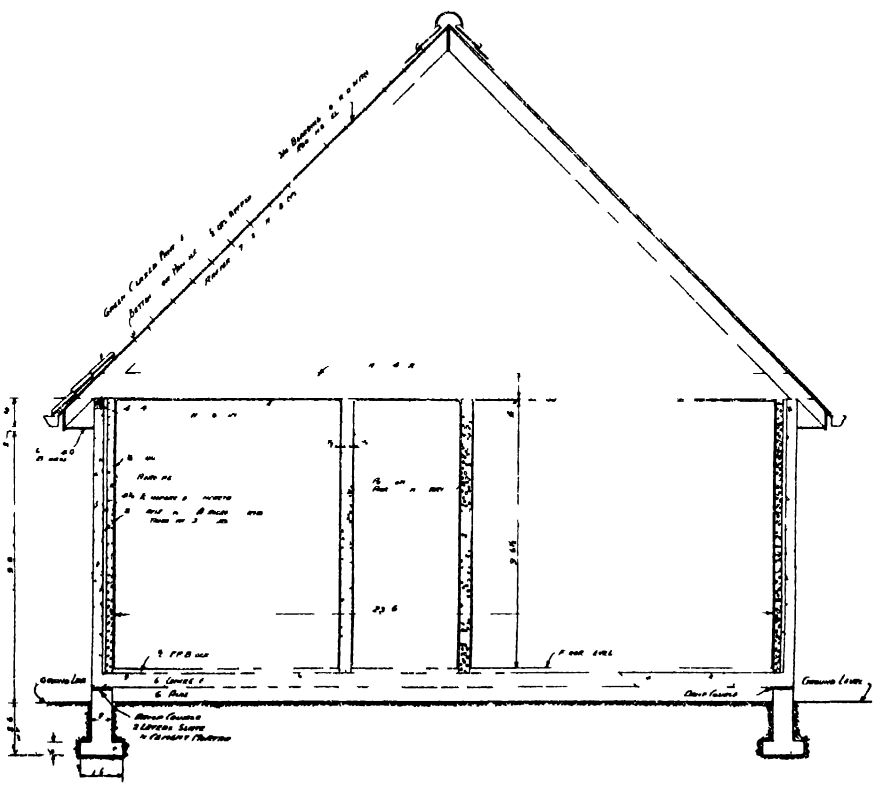
Interior of Main Hall.

Concrete Bungalow.

The concrete bungalow illustrated on page 71 has recently been completed at Freshwater Bay, Isle of Wight. The choice of materials was left entirely to the designers (Messrs. C. V. Bowles & Partners), and as the district is subject to driving mist and sudden changes of temperature it was decided to use concrete. Another factor which influenced the use of concrete was a stipulation by the client that the structure should not require repairs or decorations for many years. Cream coloured Portland cement was used for the face. A retarder was used on the shuttering, and this gave a surface resembling pebbledash.

In order to insulate the walls they were built with an inner skin of precast "Aerocrete" blocks cast on the site; this material was also used in 5-in. blocks for partition walls to make the rooms sound-resisting. The interior of the walls is rendered with cement mortar; ceilings are lined with wall boards, except in the bathroom and kitchen where white glazed asbestos sheets about 4 ft. square were used.

The chimneys and walls, which are 4½ in. thick, are of reinforced concrete throughout. We are informed that after 18 months since the walls were built there is no sign of the smallest crack or of condensation.



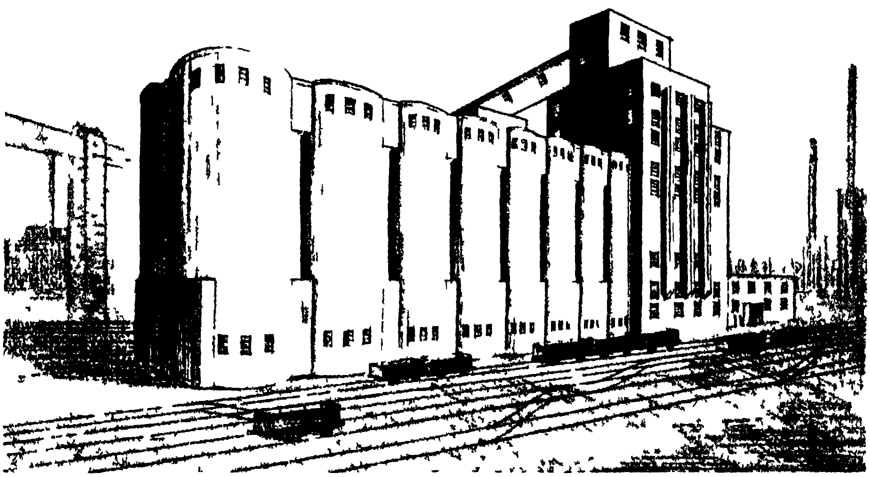
Reinforced Concrete Bungalow.

(*See page* 70)

Coal Washery and Bunkers at Cardiff.

The coal washery and bunkers illustrated are at present being erected at East Moors Works, Cardiff, for The British (Guest Keen Baldwins) Iron & Steel Co., Ltd. All the buildings in this portion of the plant will be in reinforced concrete and comprise two underground receiving hoppers, wagon hoist and tipplers, a rectangular building to house the washing apparatus, sixteen circular drainage and blending bunkers, crusher house and electrical distribution house and storage reservoirs. The underground receiving screens, conveying machinery and pumps. Above this level are two settling sumps, each of 64,000 gall. capacity, a pump sump of 68,000 gall. capacity, two dust bunkers each of 25 tons capacity, and a dirt bunker of 65 tons capacity. Above the top of these bunkers is the coal washing apparatus, and a clean water storage tank is provided on the roof.

There are sixteen circular drainage and blending bunkers each 20 ft. diameter and giving a total storage of 3,000 tons of washed coal. The bottom of each



hoppers each has a capacity of 40 tons. A pit is provided under the hopper to accommodate the discharge conveyors, and a tunnel accommodates the feed conveyor leading to the washery. The structure supporting the wagon tipplers, including the housing for the winch and motor, is constructed in reinforced concrete, and this structure is continued to form a supporting structure for the wagon hoist.

The washery building is approximately 49 ft. 6 in. long by 46 ft. 6 in. wide by 106 ft. high. The lower portion of the building accommodates the dewatering compartment is conical in shape and provided with a single opening to discharge by feeders on to the collecting conveyors. The bunkers are supported internally on hexagonal columns and externally on the circular walls which are carried to the ground level for this purpose. These walls are continued above the bunkers, and a concrete roof forms a housing for the distributing conveyors and troughs.

The coal crushing and electrical distribution house is 55 ft. 6 in. long by 30 ft. 6 in. wide and 30 ft. in height. This building is divided into two com-

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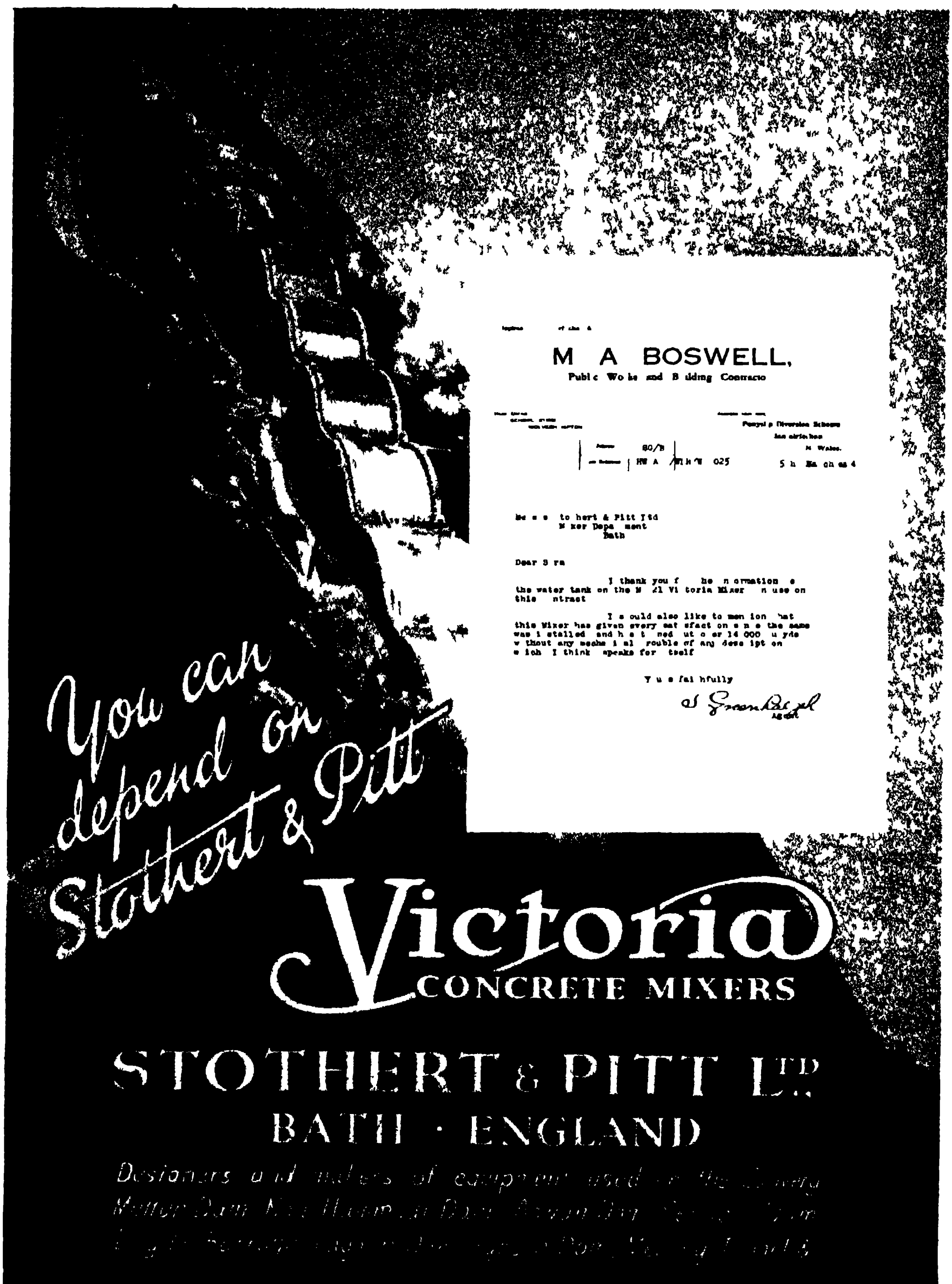


partments by a division wall in order to separate the crushers from the electric motors and apparatus. Two pairs of large double doors and special lifting beams are provided for the introduction of the machinery.

The storage reservoirs have a total capacity of 156 000 gallons and are divided into three compartments; there will be an access outlet trough running the full length and a pump sump and pit to accommodate the pump. The whole of the reservoirs and sumps will be constructed below ground level in order that any surplus water from the washery can be drained into them by gravity. The circular bunkers will be constructed by continuously moving forms, but the other structures will be carried out by ordinary methods.

The ground under the site is waterlogged and consequently all pits and sumps are designed for external water pressure and are heavy enough to prevent floating. All roofs will be flat of the slab and beam type. The windows will have reinforced concrete frames to avoid corrosion caused by gases from the coke ovens.

The reinforced concrete work is designed and being constructed by Messrs. Simon Carves Ltd., who also designed and are supplying the whole of the conveying and coal washing apparatus.

Covered Reservoir at Wealdstone.

THE reservoir illustrated has recently been completed for Messrs. Kodak Ltd. at their works at Wealdstone. It has a capacity of 400 000 gallons, the roof slab being at ground level. The columns and foundations have been designed to allow for the construction of a three-story building over the reservoir, the ground and first floors to carry 2 cwt. and the second floor $1\frac{1}{2}$ cwt. per foot super. Some trouble was experienced with the foundations due to the presence of a filled-in pond at one end of the site, and here it was necessary to deepen the foundations and suspend the floor slab. The roof slab has been designed on the flat slab principle according to the new Code of Practice.

The design and construction of the reinforced concrete work were carried out by Messrs. Brettum & Partners. The excavation was commenced during the last week in July last and the reservoir was filled for use during the third week in October. The three-story building over the reservoir was commenced at the beginning of December and is now nearing completion.

Surface Treatment of Concrete.

PRACTICAL HINTS AND DETAILS OF COSTS.

THERE is a close but not very obvious connection between the detail design of a concrete bridge, factory, etc., and the type of surface finish to be adopted, since some of the latter definitely require that the reinforcement should have more cover of concrete than is required with others. Particularly on the soffits of beams and slab in bridges over waterways, an extra ½ in. of cover beyond the customary allowance is almost essential when bush hammering or acid treatment, however carefully applied, is proposed. Nominally the cover on slab reinforcement is ¾ in., but in practice there is a risk of this being reduced during concrete placing by disturbance of the blocks, stones or other means of keeping the bars above the centering. The better and more energetic the tamping the more danger there is of disturbing these.

Bush Hammering.

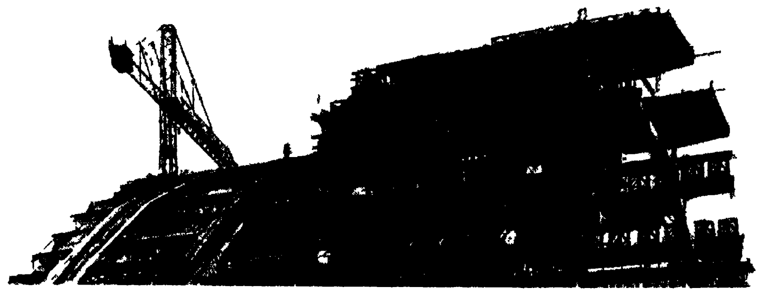
The advisability of removing the surface from thin reinforced concrete walls exposed to the weather may be questioned on the ground that the surface skin forms a waterproof layer the removal of which may allow moisture to reach the steel where it is close to the surface. The possibility of weakening a structure from this cause would be greater if heavy bush-hammering were adopted with consequent fracturing of the aggregate. This point does not arise in mass concrete work or in reinforced concrete work where the steel is well covered, and permeability of a surface treated in this way could be reduced by using a richer concrete mixture or one containing a greater proportion of sand. The slight permeability of the surface resulting from the removal of the skin of cement has, however, an æsthetic value in that concrete so treated weathers in the same way as a hard stone or granite, and does not become streaky as a cement surface so often does.

The comparative costs of bush hammered concrete and dressed natural stone are of interest. In a mass concrete arch the concrete costs about 40*s* a cubic yard, or 1*s* 6*d* a cubic foot in place, to which has to be added the cost of bush-hammering. Natural dressed stone, on the other hand, costs about 18*s* a cubic foot in place; if the dimensions of the stone are 3 ft long by 1 ft 6 in wide by 15 in thick and the thickness of the dressed face, measured over the drafted margins, is 2 in, there is a backing 3 ft by 1 ft 6 in by 13 in which has to be paid for at 18*s* a cubic foot. Considering a bush hammered pre cast concrete block of the same dimensions, the cost of the backing is only 1*s* 6*d* a cubic foot, so that the saving by using concrete is very evident. No question of strength arises.

Electric or pneumatic drive is generally used for power operated bush hammers. In the latest models the hammer is in the form of a disk, which reduces the cost of renewals when the teeth become worn. A typical electrically driven tool weighs about 14 lb and has a disk measuring 1½ in diameter with 24 cutting teeth. Each disk will treat 10 or 12 yards of concrete, and the cost of the replacement is 1*s* 6*d*. With this type of tool which costs about £30 a man can treat up to 2 sq yd per hour, cutting the face of the concrete to a depth of about $\frac{1}{16}$ in. If less penetration is needed, or if the concrete can be treated at an early age, a lighter and cheaper tool can be used which will cut the face of the concrete to a depth of about $\frac{1}{32}$ in., that is, it only removes the surface skin.

Bush hammering has sometimes been executed a fortnight after placing the concrete, but the generally adopted minimum time is three weeks. In practice the work cannot be done economically until all shuttering is stripped, which may be a matter of several months. It is uneconomical to do the bush-hammering in small areas as the shuttering is taken down.

At Wansford Bridge the aggregate was obtained on the site from a glacial drift of flint and limestone, and the maximum size permitted was 2 in instead of the customary ¾ in. The effect of using this aggregate was that the surface had the appearance of being inlaid after it had been bush-hammered, some of the large limestone pebbles being broken so as to expose their unweathered interior to contrast with the rounded aggregate exposed on removal of the cement film.



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The foundations and for that matter the whole contract were carried out by ENGEMA licensees for the Rotinoff system in Belgium

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Scrubbed Surfaces.

The simplest and cheapest method of exposing the aggregate is to wash off the cement with a stiff fibre brush and plenty of water as soon as the concrete is hard enough to permit this being done without loosening or picking out pieces of aggregate or pockets of sand For brushing hardened concrete wire brushes with flat bristles, such as are used by butchers for scouring blocks, may be employed These have the advantage that the bristles pass through holes in a movable plate and may be adjusted to give greater or less stiffness, as required according to the hardness of the concrete, by shortening or increasing the effective lengths of the bristles

On the new Twickenham Bridge over the river Thames the bush hammered surfaces of the arches have a coarse texture, and reveal the pit ballast of which the concrete was made The coping, the walls of the approaches, and the terminals to these walls and other details were pre-cast While it was desired that the pre cast units should be generally similar to the bush hammered work the opportunity was taken of using a different texture so that the pre cast work would give relief to the bridge proper The aggregates in the pre cast work consist of pit shingle and sand of the same general colour as the aggregates used for the bridge but a different texture has been obtained by the use of ⅜-in shingle to contrast with the ¾ in ballast used for the bridge A further difference has been obtained by scrubbing the faces of the pre-cast units, as this process results in a smoother texture than the bush-hammered bridge elevations The slabs were scrubbed with a wire brush within 12 hours of casting the stone

The cost of exposing the aggregate by brushing with water depends upon the hardness of the face when the work is undertaken and whether plane surfaces or work such as bridge balustrades is to be treated From 2*s* to 5*s* per square yard may be taken as outside labour costs for this class of work

If when the forms are stripped the face of the concrete is too hard to permit exposure of the aggregate by scrubbing with water, the cement may be removed with a solution of hydrochloric acid A solution of one part hydrochloric acid to six parts water should first be tried, and if this is ineffective the solution should be strengthened until copious applications of the solution and brushing with a stiff brush remove the cement with reasonably vigorous brushing As the hardness of the cement depends upon the age of the concrete, the conditions under which it has been cured, and other factors, no definite proportions of acid and water can be laid down as being effective in all cases, it is best first to try a small patch with a weak solution and strengthen it as necessary The effect obtained by the use of acid solution is similar to that obtained by scrubbing with water only The solution disintegrates the cement so that it can be brushed off, and if a uniform surface is desired care must be taken to remove the cement to the same depth over the whole of the area being treated

It is necessary thoroughly to wash from the concrete all traces of the solution after it has served its purpose or it will continue to act on the cement and to disintegrate the face of the concrete, resulting in pieces of aggregate becoming loose and falling away An alkaline solution is sometimes brushed over the acid treated surface in order to neutralise any free acid remaining on the concrete and prevent further chemical action Acid treatment is undesirable on reinforced work with small cover or low weight concrete, and acid must not be used if the aggregates are limestone Care must be taken to prevent the hands coming in contact with the solution and it is advisable to wear rubber gloves when using it

Acid treatment of concretes of different ages will result in different degrees of finish generally it is impossible to follow up the form stripping by acid treatment and the latter has to be executed at one time with consequent variations in the results unless the workmen scrub the older concrete more than the new with a view to obtaining uniform results The only additional cost of the acid treatment compared with scrubbing with water is the cost of the acid

Cement Retarders.

Retarders, which are in the form of liquid or jelly, are brushed over the forms before the concrete is placed and

retard the setting of the surface cement so that it may be easily brushed off. If the forms are removed within a week or so it is found that the surface cement is powdery and can be removed dry with a brush. Retarders are available of varying strengths which penetrate the face of the concrete to different depths so that predetermined textures can be obtained.

Special Faces applied with Sliding Shutters.

To overcome the difficulty of rendering concrete work satisfactorily a sliding plate, fitted with angle iron strips riveted to one side to give the required thickness to the mortar surfacing layer, has been developed; this enables a good bond to be obtained between a surfacing of white or coloured concrete and a grey concrete backing. An example of the application of this method in engineering work occurred in the construction of the stepped downstream face of the Ryburn Dam. In this case a steel plate shutter 2 ft. 6 in. deep provided with lifting hooks was used to separate the facing concrete from the backing. Wooden spacers notched to fit over the top edge of the steel plate were used to retain the plates at the correct distance from the timber panel forms.

When a finish of fine concrete is to be applied integrally with a backing of coarser grey Portland cement concrete in reinforced work, it is important to make the thickness of the former sufficient to give ample room for the shutter to slide without fouling the reinforcing bars. A layer less than 1 in. thick is difficult to place in this manner and calls for special care and supervision. Where smaller thicknesses have been tried the ultimate cost due to increased labour charges has often exceeded the additional charge which would have arisen by using a thicker coating of the more expensive concrete. For work which is to be lightly tooled a thickness of 1 in. is satisfactory if the surface concrete is carefully placed.

In using the sliding shutter the level of the fine facing concrete should be kept at least 2 in. or 3 in. above that of the backing so as to prevent the latter penetrating to the face. At the ends of the shutter the facing concrete should be filled into the formwork, extending around the ends of the steel plate into the grey concrete behind. This is the only satisfactory method of keeping the two separated on the surface. It requires more fine concrete than the net quantities taken off the drawings show, and an ample allowance for this "waste" must be made when tendering.

Experience has shown that surfacing can be deposited in 12-in. depths before lifting the steel separating shutter. The plates must be carefully washed and scraped after use to ensure a good finish.

The consistency to which the white facing concrete is to be mixed is best determined by trial. The considerations governing the consistency are practical rather than theoretical; if the mix is very wet its capacity for preventing penetration of the grey concrete backing is reduced and patches of dark concrete may be formed on the finished surface. On the other hand the use of an excessively dry mix is almost bound to be accompanied by fractures in the surface layer when the shutter is lifted. These also cause penetration by the grey concrete, and necessitate ultimate cutting out and patching of the surface. The latter must be avoided at all costs if the final appearance of the surface is not to be marred, since it is almost impossible to make an invisible patch on concrete work.

From the theoretical point of view there would appear to be a disadvantage in making the facing and backing concretes of different consistencies, as this would result in unequal shrinkage of the two layers during the period of hardening. This, however, is not a very important point, and practical experience has proved that satisfactory results can be obtained with different consistencies in the two layers.

For this class of work provision must be made in the design to facilitate placing the two layers of concrete. It is, for example, better to place the horizontal bars in the wall behind rather than in front of the vertical reinforcement so as to prevent them fouling the steel plates when these are being lifted. The vertical bars also act better as a guide to keep the thickness of the facing concrete uniform.

Some methods of detailing and fixing the vertical reinforcement obstruct the travelling plate and should be avoided. For example, a structure which comprises

Tempered Presdwood forms provided the beautiful finish to this concrete Pulpit with Lectern to correspond at Bishop Andrewes Church St Helier Surrey
Architect Geddes Hyslop Esq A R I B A
Contractors H M Nowell Ltd

To be continued——

The success of Tempered Presdwood as a form lining is no temporary achievement, neither is it limited to any particular type of concrete construction

It is in continual demand by contractors who have proved its efficiency, and it is as suitable for fine architectural work as for extensive surfaces of Retaining Walls, Swimming Pools, Seaside Promenades, etc.

Tempered Presdwood imparts to poured concrete a surface free from board marks and other irregularities. It provides the requisite smooth finish direct from the shuttering without the aid of rendering or other surface treatment.

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TEMPERED PRESDWOOD FORM-LINING

FOR A SMOOTH FINISH, *WITHOUT* RENDERING

a vertical wall and a sloping slab roof should not have the continuity bars bent over before the sliding shutter reaches the top of the vertical wall. If this is done these bars prevent the shutter being raised to finish the vertical surface. The bars may be bent when the latter is completed and the concrete has hardened, this being a case when the standard practice by which bars are not bent in position may be abandoned.

Experience shows that success in obtaining a good face depends on the accurate grading and even mixing of the facing concrete; on placing the concrete in a fairly dry state so that the larger stones do not tend to sink to the bottom on even tamping in small lifts; and on removing any scum which may rise to the top at the completion of each lift, or particularly at the end of each day's work. The first layer after a stop should be placed slightly wetter and very thoroughly tamped.

The face centering should be removed as early as possible and the concrete treated with a wire brush to give the desired texture. If any blemishes occur they should be filled, immediately after the concrete is brushed, with the screenings from the dry concrete according to the gauge required.

This method was used at Atcham Bridge built over the river Severn for the Shropshire County Council, the whole of the elevations being finished to resemble the local Grinshill stone. The special face, of a depth of 2 in., is composed of 3 parts ½ in. to ⅛ in. Pontesbury quartzite, 1½ parts Criggion stone 3/16 in. to dust and Leighton Buzzard sand in equal proportions, and 1 part coloured Portland cement. The surface was bush hammered with pneumatic hammers at periods varying from one month to six months after the concrete was placed. The cost of labour was 1s. 1d. an hour, and the contract price for bush hammering was 3s. per square yard for all surfaces. Since that date the contract price for bush hammering on a smaller bridge in the same county was 2s. 3d. a square yard for flat surfaces and 3s. a square yard for mouldings and copings.

In a recent case where a steel shutter was used on balcony walls to separate a white Portland cement concrete facing from a grey concrete backing, the front and back forms for the wall consisted of waterproof plywood panels ⅝ in thick with stiffened edges, some of which were used as many as fifty times before being scrapped.

Pre-Cast Facing Slabs.

If pre-cast concrete slabs are used to face a reinforced concrete structure economy can often be effected by using the slabs as shuttering. An outstanding example of this method of construction is the Dorchester Hotel, London, where cream coloured slabs were used. The external walls are 11 in thick overall, comprising a 2 in facing slab, 7 in of reinforced concrete, and a 2 in lining of cork insulation. The slabs and cork lining were used as external and internal shuttering respectively. The slabs are 2 in thick, comprising an outer face ½ in thick of a mixture of crushed cream-coloured marble and cream coloured Portland cement mixed in the proportions of 1 part cement to 2½ parts aggregate and 1½ in of concrete made with ordinary Portland cement and shingle. Reinforcing wires projecting from the backs of the slabs hold the slabs firmly to the poured concrete wall. The slabs were made face down in steel moulds and afterwards polished with carborundum disks. After three years of exposure to the London atmosphere the mortar joints between the slabs showed signs of discoloration and some of the slabs had lost a little of their original brightness. It was therefore decided to clean the whole of the elevations of the building with electrically-operated polishing machines with fine grade disks, similar to those used for cleaning down the slabs before the hotel was opened. With a view to preventing future discoloration, after the walls were cleaned down they were coated with cellulose.

In constructing the side walls of the open air swimming pool at Epsom for the Royal Automobile Club, pre-cast concrete slabs were used as shuttering

for the inner face of the reinforced concrete wall and were supported by a light timber framework with raking bracing. In this way the cost of sheeting for these forms was saved. On the outer side of the wall the shuttering consisted of timber panels strutted off the side of the excavation. When fixed in position the slabs were polished by electrically driven machines. The slabs were manufactured on the site and are 2 in. thick and comprise 1½ in. of ordinary concrete backing with a ½ in. facing of a special mixture of calcined flint aggregate and coloured Portland cement supplied ready mixed by the cement manufacturers. In order to reduce the amount of labour required in polishing, the slabs were rubbed down with newspaper before the concrete hardened appreciably; it was found that this was superior to the ordinary practice of rubbing down with sacking.

This type of precast slab is being increasingly used for lining swimming baths due to its economy compared with other materials. A recent estimate for an open air bath 100 ft. long by 30 ft. wide included £1,376 for terrazzo, £1,852 for white glazed tiles and £2,746 for white glazed bricks.

Applied Colours.

Proprietary materials are now available in many shades which may be brushed on to concrete surfaces, and which have proved constant after many years' exposure. These colours are supplied in liquid form ready for use, and are applied with a brush. Plain stippled and other finishes are available. A notable example of this treatment is the chimneys of the new Battersea Power Station which are finished in an attractive shade of buff. It is necessary thoroughly to clean the concrete surfaces before the colour is applied, steam jets sometimes being used for this purpose. On the chimneys referred to a priming coat and two finishing coats were applied.

On the bridges recently completed at Peterborough the surfaces were colour-sprayed by means of a cement gun, using a mixture of Ketton Oolites and cream coloured cement and a consider-

able amount of experimental work has been undertaken recently on the use of the cement gun for the application of coloured cement sand mixtures. When commercial sands and grey cement are used for gunite renderings on industrial structures a cloudy or patchy surface often results due either to variations in the colour of the sand, to slight variations in the amount of water used in the mix, to the sand which rebounds from the surface being blown on to parts already rendered, or to matching one day's work on to work done the previous day. With careful selection of materials and careful workmanship this defect can be avoided, and, as on the Peterborough bridges, coloured mortar of a thickness just sufficient to cover the concrete can be applied with a uniform result. On fair size contracts the cost of spraying a white or toned facing by the cement gun should not exceed 3*d* or 4*d* per square foot, including plant, materials and scaffolding up to 30 ft. or so in height. The work can be done very quickly, and only one coat is necessary.

Coloured Cements.

The introduction of coloured Portland cements and ready mixed coloured cements and aggregates during the past few years has considerably simplified the production of coloured concrete, but if the colour is to be added by the user care must be taken to ensure its even distribution if a patchy result is to be avoided. The cement and colour must be mixed in the dry state, preferably in a ball mill; failing this, the cement and colours should be passed through a fine sieve until they are uniformly distributed. Only the best metallic oxides should be used, and as these have a weakening effect on the concrete they should not be used in proportions exceeding 15 per cent. of the cement content; a smaller proportion, say, 10 per cent., is desirable, and it will often be found that a desired shade can be obtained by the use of smaller proportions of darker shades than if one shade only were used throughout. When matching colours it is advisable to mix a small batch, keeping a careful record of the quantities of materials and colour used, and make a trial slab so that the shade may be noted after the slab has dried out. If this is not the exact shade desired, the proportion of colour should be varied in other trial slabs until the desired shade is obtained after the slabs have dried out.

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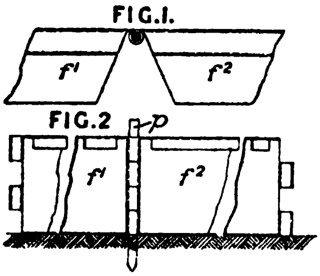
Recent Patents Relating to Concrete.

Edge Moulds for Concrete Road Construction.

395,442.—Grant, A., Arnwood, Kingscroft Road, Leatherhead. Feb. 20, 1932.

Panel moulds (f^1 f^2), in short lengths, are provided with interfitting hinge portions on their ends through which an



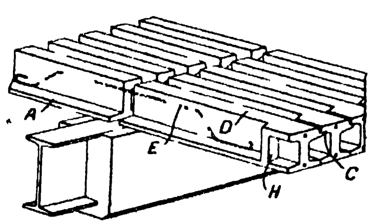
EDGE MOULDS FOR CONCRETE ROADS.

anchoring pin (p) passes to position and secure the moulds. The hinge portions are so shaped that a continuous moulding surface is presented whatever the angle the moulds are disposed at to each other in forming curved edges.

Floors and Roofs.

398,506.—Rapid Floor Co., Ltd., 59, New Oxford Street, London. (Schlagintweit, F., 7, Kronprinzenstrasse, Baden Baden, Germany.) March 11, 1932.

Relates to a floor, roof, or like construction employing pre-cast beams (A)



FLOORS AND ROOFS.

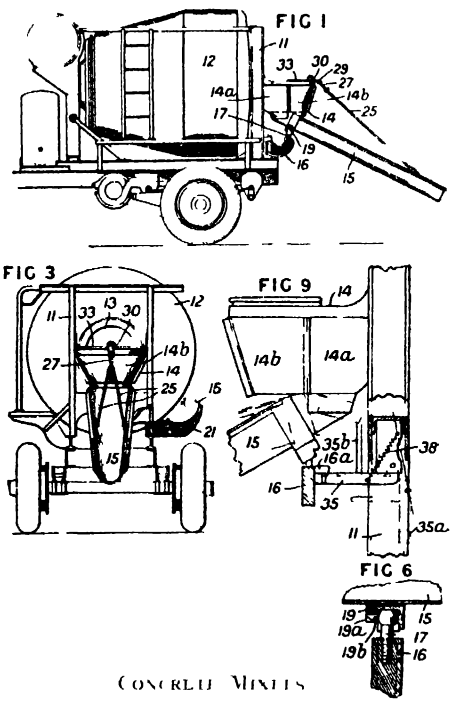
placed side by side and consists in making provision for the negative bending moments which occur at supports or fixed ends in continuous constructions by omitting part or the whole of the upper flanges of the beams towards the ends of the spans in order to provide spaces into which reverse bending moment reinforcement (E) is placed and concrete cast. The beams may be of I section, as shown in the figure, with the upper flanges (C) omitted on one side of the beam along the portions (D). The concrete filling extends to the upper surface of the beams. Stop members (H) to restrict the passage of the concrete may be cast integrally with the beams or formed separately of concrete or timber. The length of each portion (D) should not be less than one-sixth of the span. One flange of each I beam may be formed as a ledge to support the flange of the adjacent beam. In the case of beams of hollow box section, slots are formed in the top flanges for the insertion of the reinforcement and concrete.

Concrete Mixers.

400,342.—Potter, E., 3, Staple Inn, High Holborn, London. (Chain Belt Co., Milwaukee, U.S.A.) Jan. 22, 1932.

Means for distributing concrete from a concrete mixer (12) consists of a delivery chute (15) adapted to receive concrete from a discharge opening in the mixer and arranged so that it is rotatable into an inoperative position wherein it lies substantially parallel to the longitudinal axis of the mixer and adjacent thereto as shown in dotted lines in *Fig.* 3. The chute (15) is mounted at its receiving end on a universal pivot (17) on an arm (16) which is pivotally connected to the frame so as to have substantially horizontal swinging movement. The free end of the chute is supported by a rod or cable (25) connected at its upper end to a strap (27) having a series of apertures for the reception of a bolt (29) carried by a trolley (30) adapted to run on a track (33) on a delivery hopper (14). This hopper (14) is mounted adjacent the discharge opening (13) in the mixer and above the receiving end of the chute (15), and comprises side plates (14*a*) and, if necessary, a curved end member (14*b*). The track (33) consists of a pipe welded or otherwise secured to the upper edge of the hopper. The socket member (19) of the universal pivot (17) is cut away as at 19*a*, *Fig.* 6, to permit the chute to be tilted laterally when in the inoperative position, and a bolt or key (19*b*)

is provided to prevent unintentional separation of the parts of the joint. In the inoperative position the free end of the chute is supported by an arm (21) rigidly mounted on the frame (11) as shown in Fig. 3. One or more stop members (35, Fig. 9) are provided to prevent the arm (16) and the receiving end of the chute from coming into contact with the drum (12) or the frame members (11). The stop member (35) consists of an arm pivoted to the frame (11) and



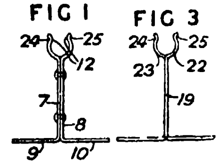
Concrete Mixers.

held in a horizontal position by a spring (38). When the receiving end of the chute (15) is in proper position beneath the hopper (14) latch members (16a, 35a) carried by the arms 16, 35 respectively engage in order to prevent the receiving end of the chute being displaced relatively to the hopper (14). The arm (35) is provided with an upstanding rod (35b) adapted to be engaged by the receiving end of the chute as it swings downwardly so as to depress the arm (35) against the action of the spring (38) and release the engagement between the latch members (16a, 35a). The arm (16) may then be swung about its pivot.

Reinforced Concrete.

400,948.—Dunker, J. H., Mole House, Esher Road, Walton on Thames. May 2, 1932.

A supporting member for positioning reinforcing mats or bars within concrete structures comprises a base member having an upstanding projection the upper end of which is provided with a fork adapted to engage by snap action the portion of the mesh or bar to be supported. The fork is adapted to be opened out in the process of inserting the mesh or bar but tends to return to its original position after the mesh or bar has been inserted. As shown in Fig. 1 the member comprises a pair of strip metal members (7, 8) having their lower ends (9, 10) bent outwardly to provide a base member and with their main parts connected by rivets, bolts, welding, etc. The upper



Reinforced Concrete.

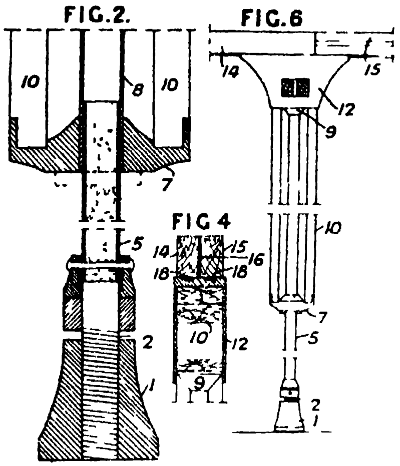
ends of the strips are opened out to provide the fork (12). In a modification the support is made from a single strip (19, Fig. 3) having its upper end divided to provide three parts of which two parts (22) are bent in an opposite direction to the other part (23) in order to provide a fork. The upper edges of the fork are bent back as at 24, 25 to facilitate entry of the reinforcement into the fork, and also to permit where desired a securing wire to be wrapped around the ends of the fork. The securing wire may alternatively be passed through holes in the supporting member or completely around such member. The forked members may engage only partially a reinforcing bar.

Shuttering Supports.

403,279.—Valentine, C. R., P.O. Box 451, Vepery, Madras, and Greaves, J. H., P.O. Box 63, Bangalore, Mysore, India. April 24, 1933.

A support for shuttering of the type comprising adjustable horizontals and a telescopic prop having rough and fine adjustment has a frame carried by shoes slidably adjusted on a tubular upright

which has a fine adjustment in relation to a metal base The fine adjustment is provided by a screw (2) in the base (1), the screw supporting a concrete filled



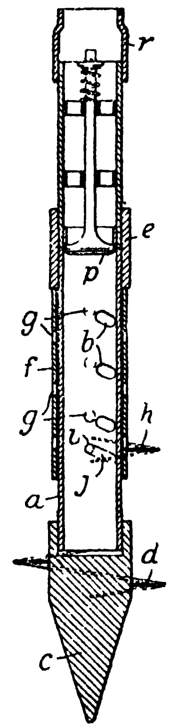
SHUTTERING SUPPORTS

tube (5) on which slides a sleeve (8) having upper and lower shoes (9, 7) respectively carrying a hardwood frame (10) and cap (12) The cap carries the adjustable horizontals (14 15) at its upper end by means of dovetail feathers (18) the horizontals also having a dovetailed sliding connection (16) to each other

Cement Guns.

405,767.—Leeds, K I N, Werrington House, Werrington Peterborough Nov 10, 1932

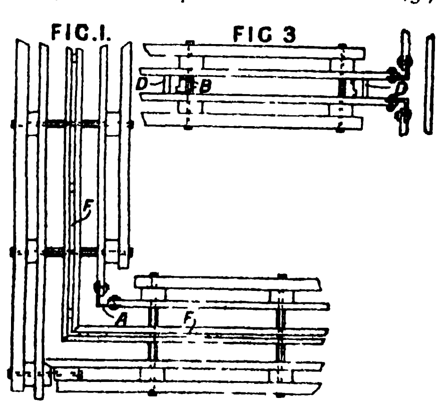
A device for introducing cement grout into substrata for strengthening them comprises a length (*a*) of tube with four equi-angularly spaced series of oblique slots (*b*), a shoe (*c*), and, at its upper end, a sleeve (*e*) Sliding on the tube (*a*) is a sleeve (*f*) with a series of round apertures (*g*), corresponding with the slots (*b*) and a helical fin (*h*) A grub screw (*i*) on the sleeve (*f*) engages in a slot (*j*) in the tube (*a*), the length of the slot being equal to that of a slot (*b*) plus twice the diameter of the screw (*i*), the arrangement being such that when the screw (*i*) abuts against the upper end of the slot (*j*) the fin (*h*) lies in the same helix as the fin (*d*) on the shoe (*c*) The device being screwed into the ground, the fin (*h*) causes the tube (*f*) to rise and to close the apertures (*b*) When the rotation is reversed, the apertures (*b, g*) coincide, and cement grout, supplied through a



coupling (*r*) and a spring supported valve (*p*) can be pumped through them

Moulding Walls in Situ.

406,954.—Long, J W 141, Dover Road, and Whatling, J L, 15, St John's Road, both in Ipswich Oct 28 1933



MOULDING WALLS IN SITU

For moulding walls in situ wooden shutters are connected at the internal angles by angle connectors (*A*) which enter rabbets in the ends of the shutters and are bolted thereto The corners of the outer shutters are bolted together as shown Internal core shuttering (*I*) is provided to form hollow walls when required The doors and window frames are positioned on the shutters by fillets (*B* *Fig* 3) and dovetail blocks (*D*) are provided on their rear edges to lock them to the concrete The rear edges of the frames may be provided with dovetail grooves instead of blocks

Floors, Roofs.

407,579. Indented Bar & Concrete Engineering Co Ltd and Vawdrey R W, Vincent House Vincent Square London June 15 1933

A method of constructing hollow reinforced concrete floors and plane surface for supporting the main longitudinal reinforcing rods (13) In casting the floor, a core (14) provided with the stirrups (16) and reinforcing rods (13) is positioned on a layer (15) of concrete disposed on the shuttering (S) and then further concrete (17) is applied, as shown in *Fig* 8, to surround the core The other cores (14) are then successively positioned and embedded in concrete Adjacent cores may be tied together by bars (18) and additional upper reinforcing rods (19) may also be provided The main reinforcing rods (13) extend from joist to joist as shown in *Fig* 2 and the cores (14) terminate at a short distance from the joists The cores are shortened or interrupted at places where openings are provided in the floor To enable the cores to house pipes electric conduits etc they may be provided with longitudinal openings (23) formed by bending downwardly the longitudinal opposed edges of the metal to form flanges (24) as shown in *Fig* 9 and such openings are filled during casting

FIG 2

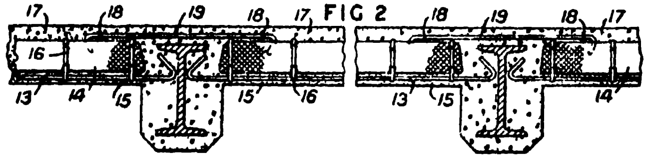


FIG 8

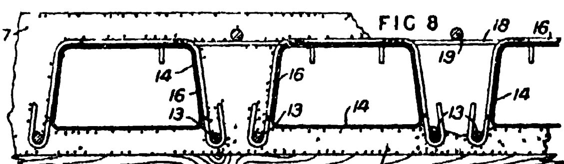
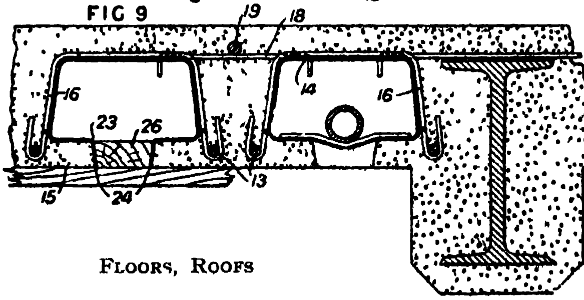


FIG 9



Floors, Roofs

roofs consists in placing tubular cores (14) of expanded metal metal mesh metal lattice, or like openwork metal in position in the structure and totally and permanently embedding it in concrete to form a hollow reinforced concrete slab the upper and lower faces of which are composed entirely of concrete Stirrups (16) are wound to each of the cores (14) with wood members (26) The pipes, etc , are inserted after the floor has been cast, and the flanges (24) are then bent upwardly and the opening (23) is filled with concrete If desired, selected portions only of a floor may be cast in the manner described, the remaining portion being formed of pre cast hollow reinforced concrete slabs

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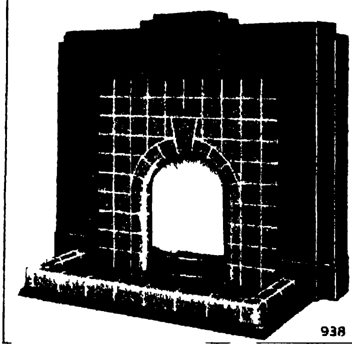

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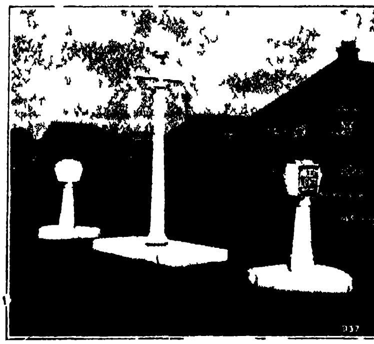

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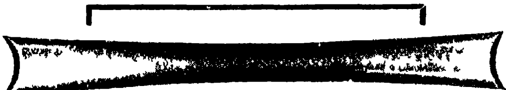

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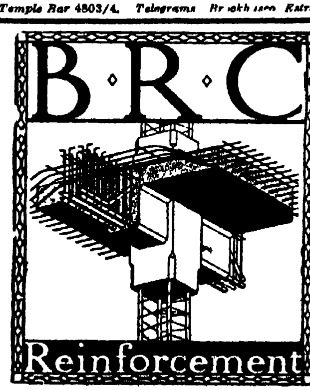


DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS

		s	d
Best Washed Sand	per yard	8	3
Clean Shingle, ¾ in mesh	,,	7	3
,, ,, ⅜ in mesh	,,	9	0
Thames ballast	,,	7	3
Broken brick (¾ in)	,,	11	3

Best British Portland Cement (delivered London area in 4 ton loads and upwards) in jute sacks or non returnable paper bags per ton 40s, 1 ton to 3 ton 19 cwt loads, 42s per ton, jute sacks charged 1s 6d each and allowed 1s 6d each if returned in good condition within 3 months

Rapid-Hardening Portland Cement delivered London 6s per ton extra

"Colorcrete" red and buff rapid hardening Portland Cement delivered London in non returnable paper bags per ton 66s

"Cullamix" Coloured Portland cement and aggregate delivered London area from £5 per ton upwards

"Snowcrete" White Portland Cement, £8 15s per ton delivered London including non returnable paper bags

"Super Cement" Waterproof Cement (paper bags free) per ton 70s

BOARDING FOR SHUTTERING.—

		Sawn	Wrot
1 in	per square	21s 0d	25s 0d
1¼ in	,,	26s 0d	30s 6d
1½ in	,,	31s 0d	36s 0d

SAWN TIMBER FOR STRUTS AND SUPPORTS

3 in by 4 in	from £10 per standard
3 in by 6 in and 3 in by 7 in	£18 ,,

MILD STEEL RODS FOR REINFORCEMENT—

		s	d
⅝ in to ⅞ in Rounds	per cwt	8	6
$\frac{9}{16}$ in to ½ in Rounds	,,	8	9
⅜ in Rounds	,,	9	0
¼ in Rounds		10	0

Breeze Slabs per yd super 2 in 1/6, 2½ in, 1/8, 3 in, 2/, 4 in 2/4

MATERIAL AND LABOUR INCLUDING 10 PER CENT PROFIT

(Based on Contracts up to £2 000)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 2 4—

				s	d
Do	do	in foundations	per foot cube	1	
Do	do	in columns	,, ,,	1	4
Do	do	in beams	,,	1	6
Do	do	in floor slabs 4 in thick	per yard super	3	8
Do	do	in floor slabs 5 in thick	,, ,,	4	7
Do	do	in floor slabs 6 in thick	,, ,,	5	6
Do	do	in floor slabs 7 in thick	,, ,,	6	5
Do	do	in walls 6 in thick	,, ,,	5	6

(Add for hoisting 3s 6d per yard cube above ground floor level Add for rapid hardening Portland Cement 2s per yard cube)

STEEL REINFORCEMENT INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s	d
From ¼ in to ⅜ in	per cwt	15	0
,, $\frac{7}{16}$ in to ½ in	,,	14	0
,, $\frac{9}{16}$ in to 2⅞ in	,,	13	0

EXTRA LABOUR TO BENDS in ¼ in rods ½d, ⅜ in rods, 1d, ½ in rods 1¼d, ⅝ in rods, 1½d, ¾ in rods 1¾d, ⅞ in rods 2d, 1 in rods, 2¼d, 1⅛ in rods, 3d, 1¼ in rods 3½d, 1⅜ in rods 4½d, 1½ in rods 6d

EXTRA LABOUR TO HOOK BENDS ¼ in, 1d, ⅜ in, 2d, ½ in, 2½d, ⅝ in, 3d, ¾ in, 3½d, ⅞ in 4d, 1 in, 4½d, 1⅛ in, 6d, 1¼ in, 7d

SHUTTERING—

		s	d
Shuttering and Supports for Concrete Walls (both sides measured)	per square	40	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft high	per square	42	0
Do do in small quantities	per ft super	0	6
Shuttering and Supports to Stanchions average 18 in by 18 in	per ft super	0	6
Do do as last, in narrow widths	,, ,,	0	7½
Do do to sides and soffits of beams, average 9 in by 12 in	,, ,,	0	7½
Do do as last, in narrow widths	,, ,,	0	8½
Raking, cutting, and waste to shuttering	per ft run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are —Carpenters and joiners, 1/7 per hour; Carpenters working on old shuttering, 1/8, Labourers on building works, 1/2¼; Men on mixers and hoists, 1/3¼; Bar-benders, 1/3¼

Prospective New Concrete Work.

AYLESBURY.—*Swimming Pool.*—The T.C. is to construct an open air swimming pool at the Vale at an estimated cost of £6,167.

BARNSTAPLE.—*Sewerage and Sewage Disposal.*—The R.D.C. has applied for sanction to a loan of £5,000 for sewerage and sewage disposal works at Combe Martin.

BERWICK.—*Swimming Pool, etc.*—The T.C. is considering the construction of a swimming pool, approach road and sewerage works at a cost of £20,357.

BILLINGHAM.—*Bridge.*—The Durham C.C. is to construct a bridge over Belasis Lane level crossing at Billingham.

BRIGHTON.—*Bridge.*—The T.C. proposes to reconstruct Dyke road Drive railway bridge at an estimated cost of £20,000.

CAISTOR.—*Water Supply.*—The R.D.C. is to construct water supply works at an estimated cost of £23,000.

CAMBRIDGE.—*Swimming Pool.*—The T.C. is recommended to construct a swimming pool at Coldham's Common at an estimated cost of £3,460.

COLERAINE.—*Harbour Works.*—The Harbour Board proposes to extend the harbour at an estimated cost of £3,000.

DORKING.—*Sewerage and Sewage Disposal.*—The U.D.C. has applied for sanction to a loan of £5,600 for sewerage and sewage disposal works.

EAST DEREHAM.—*Sewage Disposal.*—The U.D.C. has approved a proposal for reconstructing the sewage disposal works at an estimated cost of £6,400.

EDINBURGH.—*Chimney.*—The T.C. is considering the erection of a chimney at the Portobello power station at an estimated cost of £60,000.

ELY.—*Water Supply.*—The R.D.C. is to construct water supply works at an estimated cost of £37,000.

GLASGOW.—*Roads.*—The T.C. has been recommended to reconstruct Springborn Gardens at an estimated cost of £38,000.

ISLE OF AXHOLME (LINCS.).—*Water Supply.*—The R.D.C. is to construct water supply works at an estimated cost of £52,000.

LANARK.—*Bridge.*—The C.C. has approved the construction of a bridge, Millheugh, Larkhall, at an estimated cost of £4,200.

LONDON.—*Swimming Pool.*—The L.C.C. has been recommended to approve an estimate of £25,000 for the construction of an open air swimming pool at Victoria Park.

MARGATE.—*Reservoir.*—The T.C. has received sanction to a loan of £26,342 for the construction of a 5,000,000 gall. reservoir at Flete.

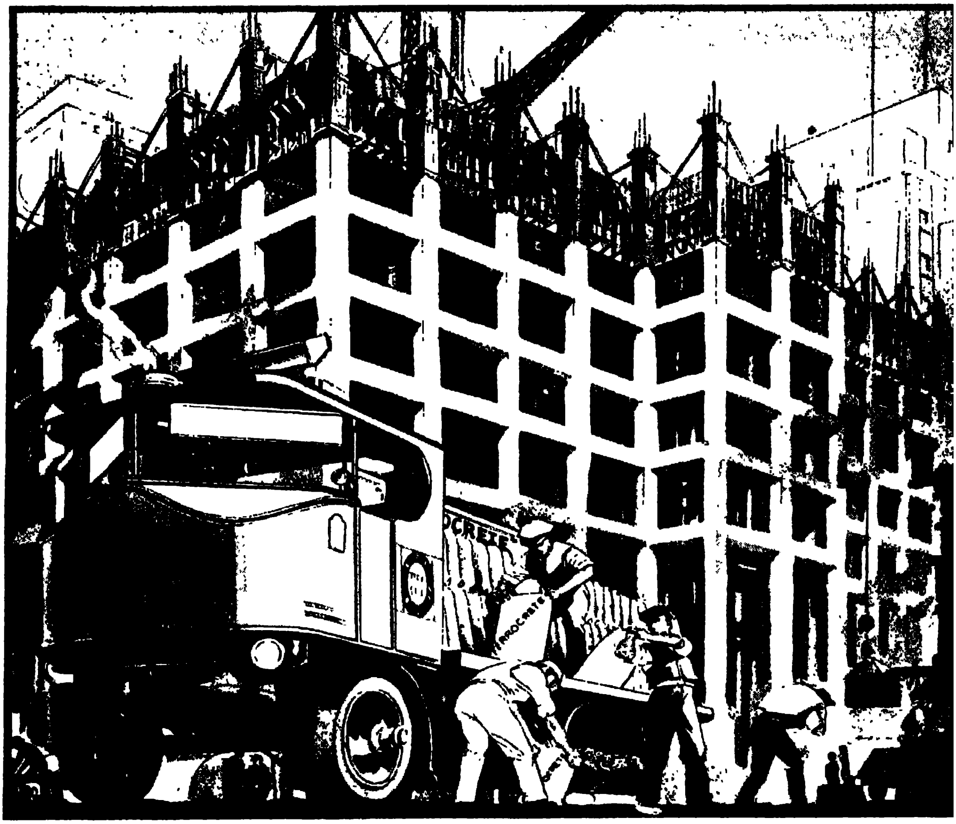
MIDDLESEX.—*Viaduct, etc.*—The C.C. is considering the completion of Western Avenue, including a viaduct about a mile long, at an estimated cost of £351,695.

MIDDLESEX.—*Bridge Widening.*—The C.C., in conjunction with the G.W.R., is to widen the bridge over the railway at Station road, Hayes, by means of steel girders carrying a reinforced concrete deck.

PENICUICK.—*Reservoir.*—The B.C. has approved the construction of a reservoir at Quarrel Burn at an estimated cost of £13,500.

PORTSMOUTH.—*Road Reconstruction.*—The T.C. has agreed to construct a road from the Guildhall to Fratton, at an estimated cost of £206,100.

RAMSGATE.—*Bathing Pool.*—The T.C. is considering the construction of a bathing pool near Winterstroke Gardens, at an estimated cost of £35,000.



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RAMSGATE.—*Sea Defence, Bathing Pool, etc.*—The T.C. is considering the construction of an undercliff promenade, sea defence works, bathing pool and shelters at an estimated cost of £60,000.

SAFFRON WALDEN.—*Water Supply.*—The R.D.C. is to construct water supply works in Hensham and district at an estimated cost of £16,500.

SEAFORD.—*Sewerage and Pumping Station.*—The U.D.C. is to construct drainage works and a pumping station at an estimated cost of approximately £16,000.

SHELLEY.—*Sewerage.*—The U.D.C. has applied for sanction to a loan of £8,000 for the construction of sewerage works.

SHIPLEY.—*Water Supply.*—The U.D.C. has applied for sanction to borrow £4,130 for water supply works.

STIRLING.—*Bridge.*—The C.C. is considering the widening of the bridge over the river Allan at an estimated cost of £11,000, or alternatively the construction of a new bridge at estimated costs of £38,000 and £41,000.

SURREY.—*Bridges.*—The C.C. has recommended the reconstruction of 16 bridges in the county.

SWANSCOMBE.—*Swimming Bath.*—The U.D.C. proposes to construct a swimming bath at an estimated cost of £6,000.

TEIGNMOUTH.—*Concrete Shelter.*—The U.D.C. propose to construct a reinforced concrete shelter on East Cliff terrace.

THRAPSTON.—*Water Supply.*—The R.D.C. has applied for sanction to a loan of £4,130 for water supply works.

WAKEFIELD.—*Swimming Baths.*—Plans are being prepared for the construction of swimming baths at an estimated cost of £25,000.

WALLINGFORD.—*Bathing Pool.*—The T.C. has received sanction to a loan of £1,075 for the construction of a bathing pool.

WALLINGTON.—*Bridge.*—The U.D.C. proposes to reconstruct the bridge over the river Wandle.

WARMLEY (GLOS.).—*Sewage.*—The R.D.C. has approved the construction of sewage works at Kingswood, Warmley, at an estimated cost of £68,000.

WEST LANCASHIRE.—*Sewerage.*—The R.D.C. has applied for sanction to proceed with the Sefton sewerage scheme at an estimated cost of £50,000.

WESTON-SUPER-MARE.—*Bathing Pools.*—The U.D.C. is considering the provision of a bathing pool in Glentworth Bay at an estimated cost of £35,000, and a swimming pool near Clevedon road at an estimated cost of £21,000.

Tenders Accepted.

ARDGLASS — *Water Supply* The Downpatrick R D C has accepted the tender of G P Trentham, Ltd, at £2,817, for works in connection with the construction of water supply works

BRIDGWATER *Cattle Market* — The T C has accepted the tender of H W Pollard & Sons, at £20,540, for the construction of a cattle market

BURNLEY —*Bridge* —The T C has accepted the tender of G Wimpey & Co, Ltd at £6,996, for the reconstruction of Mitre bridge, Westgate

GALLOWAY - *Water Supply* — The Galloway Water Power Co, Ltd has accepted the tender of A M Carmichael, Edinburgh, for the construction of a reinforced concrete pipe aqueduct and foundations, etc for the power station at Kendoon, the contract price is approximately £70,000

GRANGE —*Reservoir* The U D C has accepted the tender of Birch & Sons, Grange over Sands, at £1,091 for the construction of a covered reservoir

HARWICH —*Concrete Roads* —The T C has accepted the tender of G Wimpey & Co, at £4,196, for the construction of reinforced concrete roads and incidental works in Barrack Lane and Becon Hill Avenue

HUNTLY *Water Supply* —The T C has accepted the tender of L Flaherty, Falkirk, at £9,656, for constructional works in connection with the water supply

HUTTON —*Swimming Bath* —The Lancashire Education Committee has accepted the tender of J Turner & Sons (Preston), Ltd, at an approximate cost of £6,000, for the construction of a swimming bath at the grammar school

KIDSGROVE - *Reservoir Cover* —The U D C has accepted the tender of S & P Cope, Smallthorne, at £210, for the construction of a reinforced concrete cover for Mow Cop reservoir

LONDON —*Subway* —The City Corporation has provisionally accepted the tender of Mitchell Bros, Sons & Co, Ltd, London at £21,066, for the construction of a subway at the junction of King William Street, Cannon Street and Gracechurch Street Other tenders submitted Chas Brand & Son, £27,833, John Mowlem & Co, Ltd, £22,890, Kinnear, Moodie & Co, Ltd, £22,740, John Cochrane & Sons, Ltd, £22,410

LONDON (BERMONDSEY) —*Concrete Piling* —The L C C has accepted the tender of Simplex Concrete Piles, Ltd, at £990, for pile foundations for a block of dwellings on the Dickens' estate Other tenders submitted François Cementation Co, Ltd, £949, Holst & Co, Ltd., £950, Bierrum & Partners, £955; Concrete Piling, Ltd, £1,334; J Gill (Contractors), Ltd, £1,339, S Williams & Sons, Ltd, £1,425, Franki Compressed Pile Co, Ltd, £1,900

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